City of Waukesha, Wisconsin

130 Delafield Street │ Waukesha, WI 53188

Final Report, Phase I
Sanitary Sewer Master Plan

City of Waukesha, Wisconsin
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Donohue Project No.: 11564

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City of Waukesha, Wisconsin

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EXECUTIVE SUMMARY

This report summarizes the results of the first of the two-phase Sanitary Sewer Master Plan. This plan, once complete, will identify those capital improvement projects that Waukesha should implement to provide reliable wastewater collection and conveyance for at least the next 5 years. The Capital Improvement Plan (CIP), to be completed during Phase II, will list what specific projects should be implemented when, and at what cost. In addition, Donohue will prepare a CMOM Implementation Plan that will provide specific operational and organizational improvements to bring the City into conformance with EPA's CMOM guidelines.

This first phase has focused primarily on evaluating flows and conveyance capacities. This has been accomplished by the development of a MikeUrban/MOUSE hydraulic model. This model was calibrated to pump station and flow monitoring data collected in the spring and summer of 2009.

Flow monitoring data was also used to complete a comprehensive Inflow and Infiltration (I&I) Study. This study has determined that I&I in the majority of the service area is not excessive. However in Pebble Valley, I&I creates operational challenges with the Pebble Valley and Greenmeadow pump stations. And while it does not present any operational challenges at the moment, I&I from the Heyer Drive service area is some of the highest in the City. The older downtown sewers also contribute significant I&I; Donohue recommends that all of these sewer undergo a Sanitary Sewer Evaluation Survey (SSES).

A limited SSES was conducted in 2009 by smoke testing those areas that appeared to experience the most direct inflow. While relatively few direct sources of inflow were located, the testing did reveal that sewers tested in the downtown and Heyer Drive areas are of questionable structural integrity. Under Phase II of this project, the SSES program will be expanded to include additional smoke testing, sewer televising, and dyed-water flooding to locate sewer defects permitting the entry of clear water flows.

The Aviation Drive, Coneview, Pebble Valley, Summit, and Sunset pump stations are at risk for flooding during major rainfall events. The storm of June 2008, a 100-year event, inundated several of these stations. There are some relatively inexpensive remedies that Waukesha is considering to protect these stations from surface flooding.

The West Bypass and Southeast Bypass sewer projects under consideration could eliminate up the following eleven pump stations: Coneview, Heritage Hills, Fiddler's Creek, Summit, Tallgrass, MacArthur Rd, Pebble Valley, Heyer Dr, West Ave, Milky Way Rd, Burr Oak Blvd. Replacing these stations with gravity sewers would improve system reliability by reducing the number of stations that would have to be maintained, and would eliminate these stations' force mains, some of which have been problematic, and would reduce energy and O&M costs. Preliminary estimates to design and construct these bypasses total approximately $18M.

Several of Waukesha's force mains have leaked and/or failed; these have been repaired or replaced. The General Electric force main and 1500' of the West Ave force main are scheduled for replacement in 2010. In most cases, external corrosion of ferrous force mains has been the principal method of failure. As per EPA's request, a desktop force main risk assessment has been completed in order to prioritize all force mains in order of risk. Under Phase II of this project, physical condition assessments will be conducted on those force mains at greatest risk in order to repair/replace them <u>before</u> a failure can occur. Donohue recommends performing External Corrosion Direct Assessments (ECDA) of the following five force mains at greatest risk: West Ave. (remaining 1800'), 600' of Greenmeadow, Pebbley Valley, 1800' of Heyer Dr, and 2000' of Burr

Oak Blvd. Which, if any, addition force main testing will be required will be determined from the findings of these initial tests.

While many of the City's operations are consistent with EPA's CMOM Guidance Manuel, CMOM Program Planning has identified several areas of improvement whereby Waukesha's operations and maintenance could be brought into better conformance with EPA guidelines. Areas for improvement generally include sewer maintenance/inspection and records keeping. Under Phase II of this project, a CMOM Implementation Plan will be developed that will provide specific instructions to bring Waukesha into compliance with EPA guidelines. This comprehensive plan will focus on improved documentation, record keeping, communication, and coordination. In addition, the City has implemented a sewer televising program that will be coordinated with the sewer cleaning program to monitor sewer condition and clean/rehabilitate them in an efficient, proactive manner. Under the second phase of this project, Donohue will prepare a comprehensive CMOM Implementation Plan that once implemented, will bring the City in full compliance with EPA guidelines.

CHAPTER I – INTRODUCTION

1.1 PROJECT BACKGROUND / OBJECTIVES

The overall objectives of this two-phase project are to improve:

- System capacity,
- Efficiency, and
- Integrity.

The first phase of this project focuses primarily on the first two items; however, it has identified areas of questionable structural integrity that warrant further inspection under Phase 2.

1.1.1 JUNE 2008 STORM

On June 7th and 8th, 2008, a particularly large storm struck the City of Waukesha (City). Since there were no first-order weather stations operating within the City service area during this storm, it is difficult to thoroughly characterize the frequency and magnitude of this event. However, the gauge at Mitchell International Airport, approximately 17 miles from the City, recorded 7 inches of rainfall, with over 4 inches falling in one 3-hour period (see Figure 1).

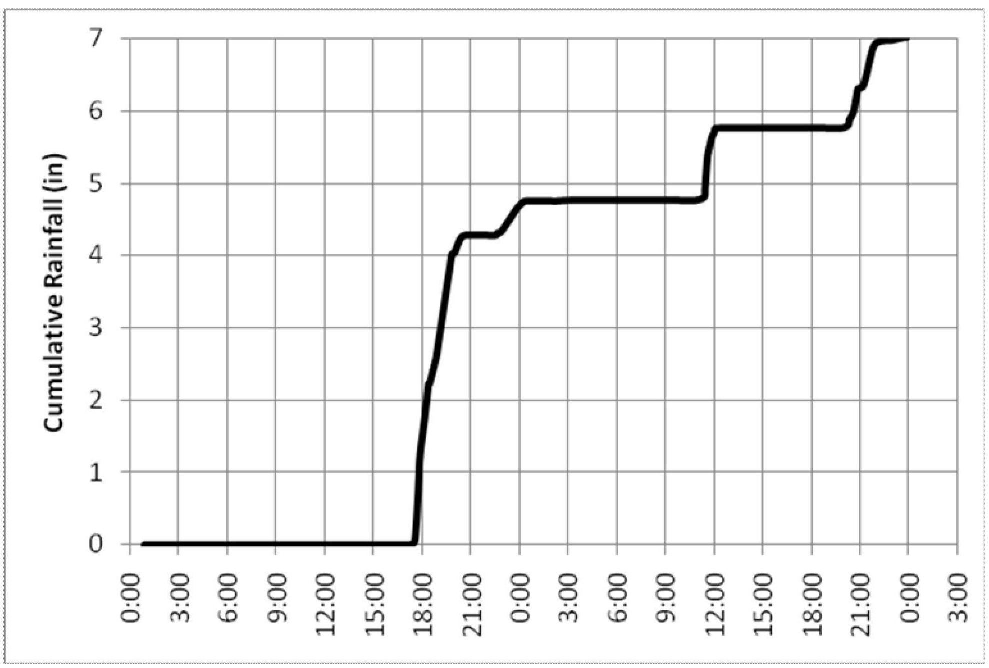


Figure 1 – June 2008 Rainfall Event

We can, however, infer the magnitude of this event from Fox River stage data provided by USGS. The maximum water surface elevation recorded at this gauge, located on the Fox River 100 feet downstream of North Prairie Avenue, was 801.82 feet. This elevation is within 4 inches of the 100-year base flood elevation documented in the FEMA Flood Insurance Study (see Figure 2).

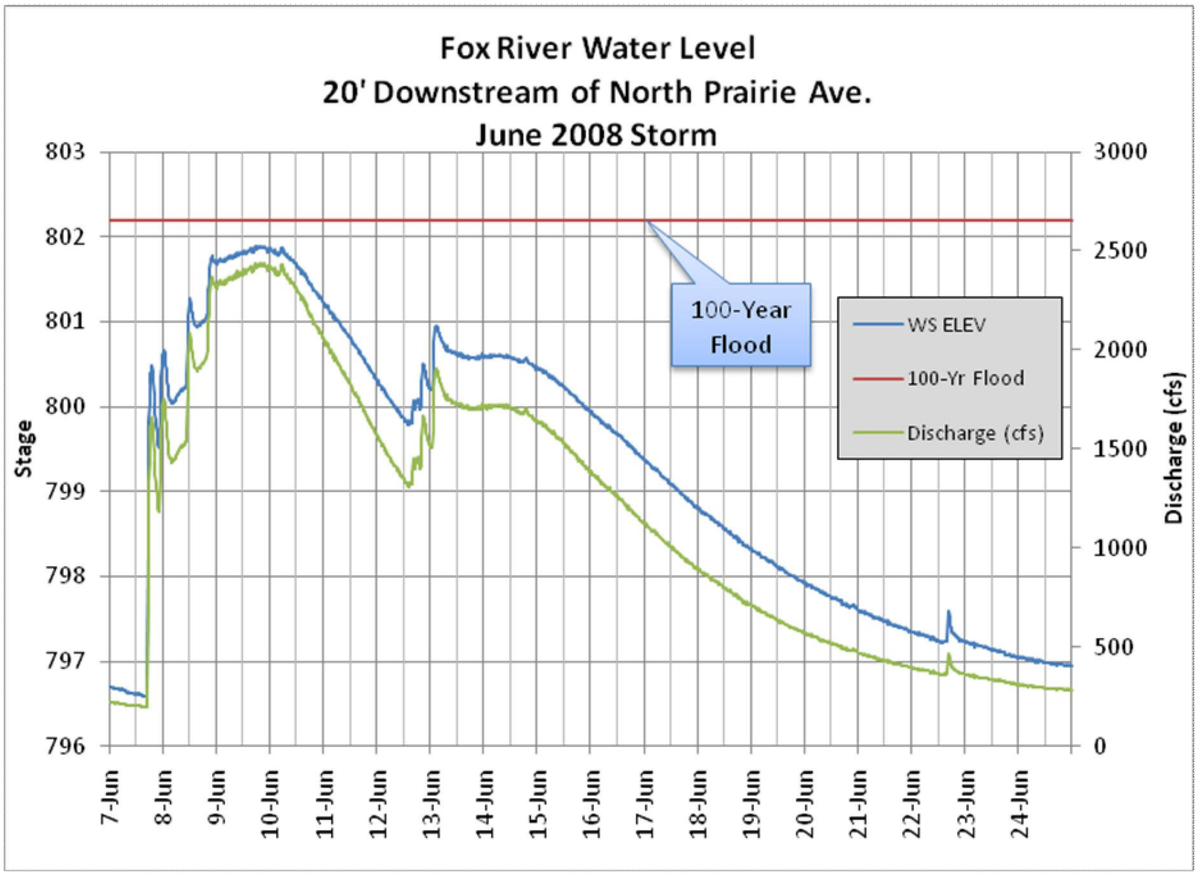


Figure 2 – June 2008 Fox River Discharge/Stage Data

This storm presented some unique operational challenges for City personnel and exposed some potential weaknesses in the collection system. Several of these are discussed further in Section 1.3. Figure 3 illustrates the impact this storm had on plant flows. Figure 4 indicates a strong correlation between river stage and plant flows which might be an indication that river water was entering the collection system. However, it is important not to mistake correlation for causation. Not only did the storm result in high river levels, but high groundwater levels, which would have resulted in increased I&I, particularly for older sewers that often cross or lay adjacent to the river. Plant personnel did, on the other hand, locate a manhole with a broken cover adjacent to the river that was allowing the river to drain into the sewer. This has since been repaired.

In addition, the peak river stage elevation of 801.82 feet is above the rims of 160 manholes along 8.5 miles of sewer adjacent to the river (Figure 5). These would likely have been submerged during this storm. Approximately half of these were inspected for missing covers following the storm, yet no defects were found. The majority of manholes adjacent to the river have been sealed while several others are scheduled to be rehabilitated as streets are reconstructed. Donohue recommends that the City confirm that all of these manholes are sealed/rehabilitated.

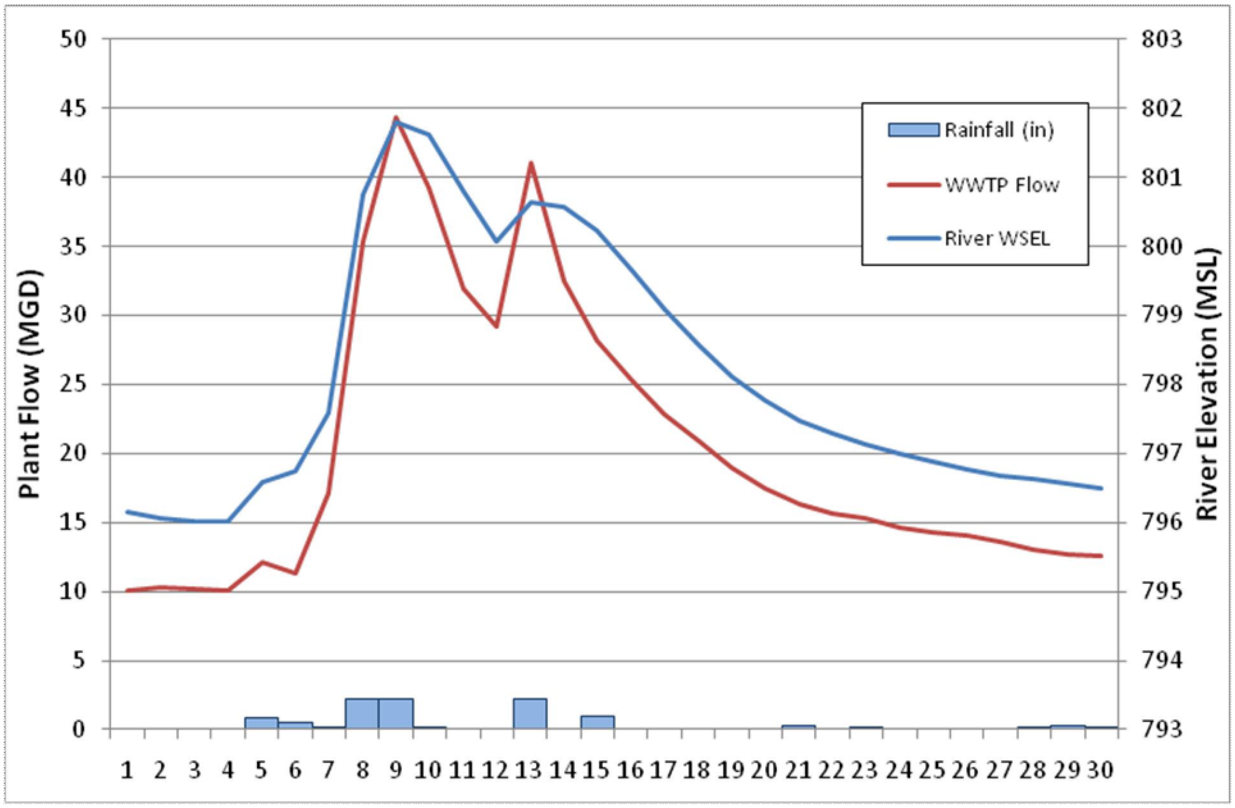


Figure 3 – June 2008 Plant Flow

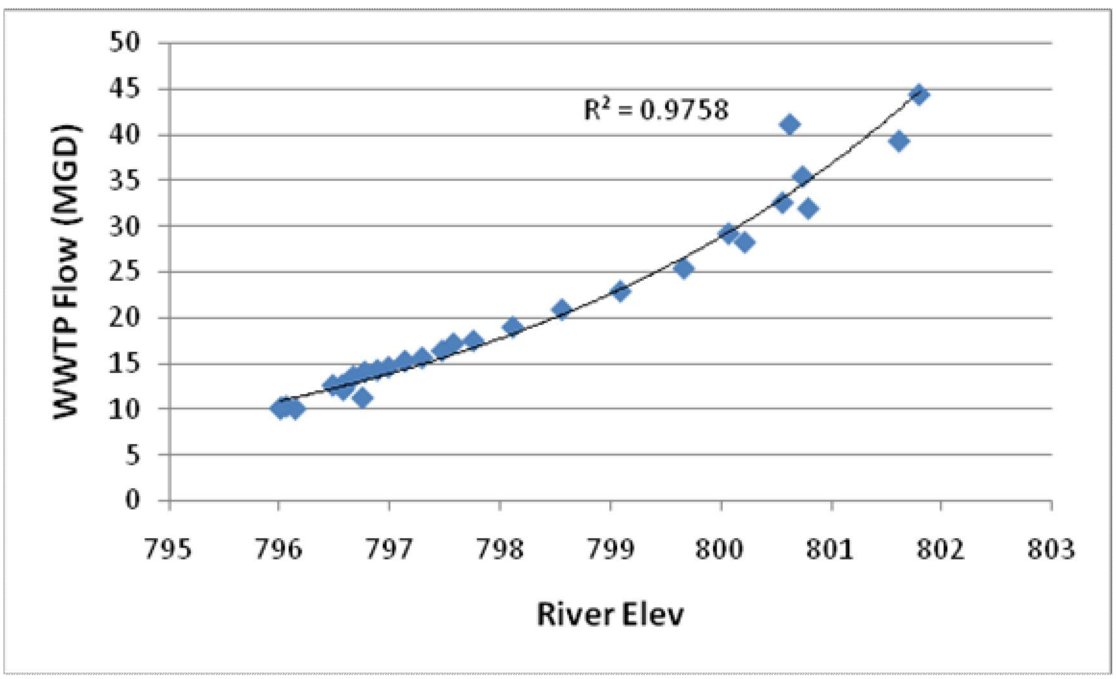


Figure 4 – Plant Flow vs. River Stage

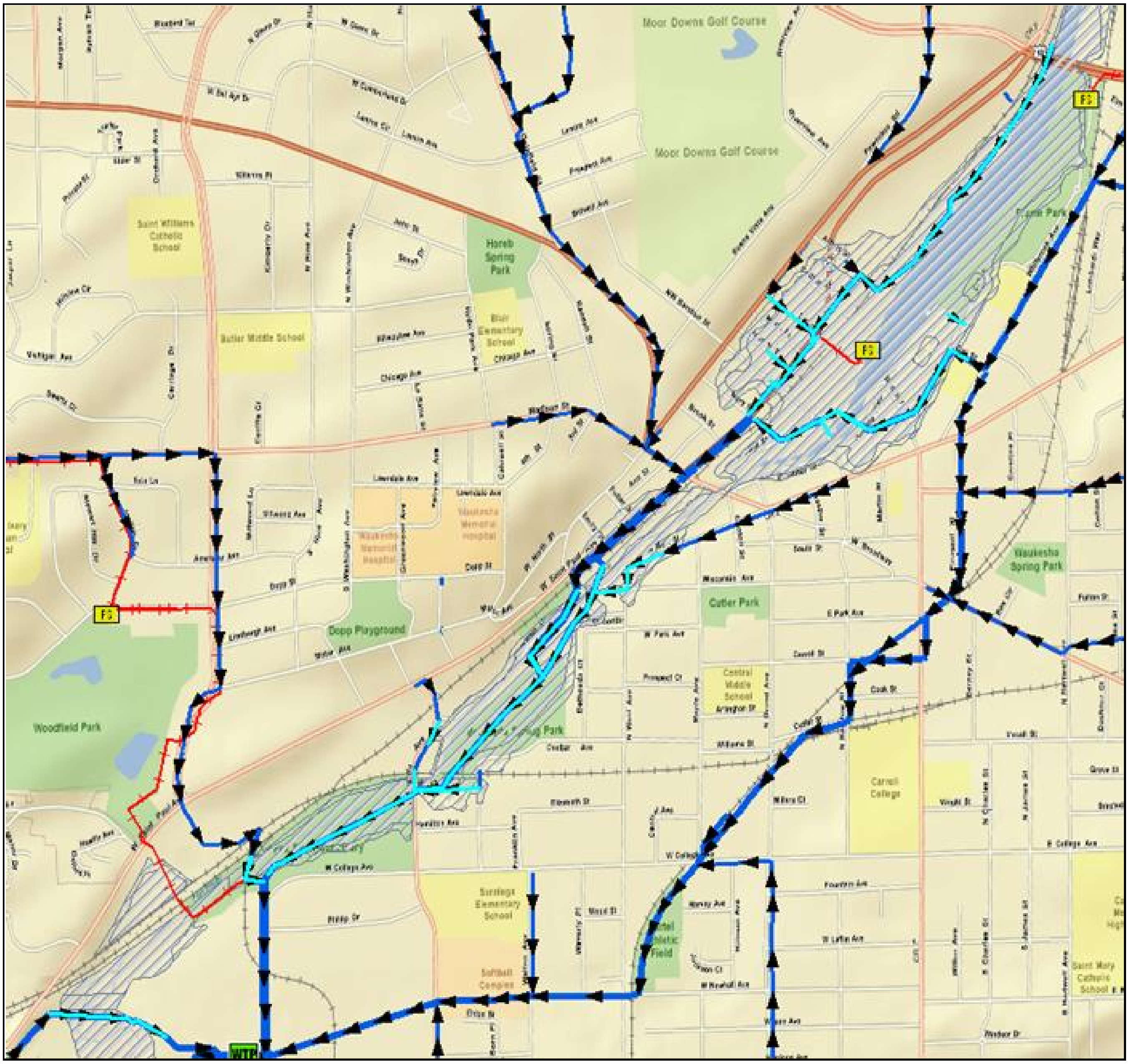


Figure 5 – Sewers within the Floodplain

1.2 COLLECTION SYSTEM SUMMARY

1.2.1 PUMP STATIONS

Of the 16,200-acre service area, only a 5,400-acre (33%) central area flows to the treatment plant by gravity. The remaining 10,800-acre area outside of the downtown core must be pumped, sometimes multiple times, before the flow can be conveyed to the plant by gravity. The City's collection system contains 13 private and 43 public pump stations. Table 1 is an inventory of all 56 pump stations.

Table 1 – Pump Station Inventory

Pressurized Service Laterals	PRIVATE - 1308 SUNSET CIR	1308 SUNSET CIR
	PRIVATE - 2715 SILVERNAIL RD	2715 SILVERNAIL RD
	PRIVATE - 2835 N GRANDVIEW BLVD	2835 N GRANDVIEW BLVD
	PRIVATE - 2903 N GRANDVIEW BLVD	2903 N GRANDVIEW BLVD
	PRIVATE - 3421-3419 RED MAPLE WAY	3421-3419 RED MAPLE WAY
	PRIVATE - 3425-3423 RED MAPLE WAY	3425-3423 RED MAPLE WAY
	PRIVATE - GOOD TIMES DAY CAMP	443 MERRILL HILLS RD
	PRIVATE - KOHLS SHOPPING CENTER	2200 W ST PAUL AVE
	PRIVATE - LANDSBERG CENTER	2700 GOLF RD
	PRIVATE - LIFECARE HOSPITAL OF WI	2400 GOLF RD
	PRIVATE - ST JOHN NEUMANN	2400 STH 59
	PRIVATE - STEINHAFELS	W231 N1013 COUNTY HWY F
	PRIVATE - WELDALL	2001 S PRAIRIE AVE
City Pump Stations / Grinders	AVIATION DR	2515 AVIATION DR
	BADGER DR	2316 BADGER DR
	BLUEMOUND RD	2332 BLUEMOUND RD
	BLUEMOUND RD WEST	332 BLUEMOUND RD
	BURR OAK BLVD	1940 OAKDALE DR
	CONEVIEW	3028 CONE VIEW LN
	CORPORATE CENTER	717 EXECUTIVE PL
	DEER PATH	1969 FOXCROFT LN
	DEER TRAILS	2107 DEER CREEK CROSSING
	FIDDLERS CREEK	3425 TURNBERRY OAK DR
	FOX LAKE VILLAGE	2922 MAKOU TRAIL
	FOX POINT	2000 FOX RIVER PKWY
	FRAME PARK GRINDER	701 E MORELAND BLVD
	GENERAL ELECTRIC	3196 N GRANDVIEW BLVD
	GOLF RD	2838 GOLF RD
	GREENMEADOW	205 GREENMEADOW DR
	HEYER DR	1215 HEYER DR
	HOLLIDALE	2218 HOLLIDALE DR
	LESLIE DR GRINDER	2408 LESLIE LN
	MACARTHUR RD	3001 MACARTHUR RD
	MADISON ST	3327 MADISON ST
	MILKY WAY RD	1601 MILKY WAY RD
	MORELAND BLVD	1440 WHITEROCK AVE
	NORTHVIEW RD	1110 NORTHVIEW RD
	PARK REC GRINDER	1900 AIRPORT RD
	PATRICIA LN GRINDER	1701 PATRICIA LN
	PEARL ST	1424 PEARL ST
	PEBBLE VALLEY	2571 PEBBLE VALLEY RD
	POLICE PISTOL RANGE GRINDER	800 SENTRY DR
	RIVER HILLS	913 DANA LN
	RIVER PLACE	2404 FOX RIVER PKWY
	RIVERS CROSSING	3555 RIVER VALLEY RD
	RUBEN DR	1800 JEFFREY LN
	SILVERNAIL	920 SILVERNAIL RD
	SPRINGBROOK	2210 SPRINGBROOK N
	SUMMIT AVE	1101 MEADOWBROOK RD
	SUNSET DR	1294 W SUNSET DR
	TALLGRASS	901 WINTERBERRY DR
	UNION ST GRINDER	101 UNION ST
	WALMART	1101 STH 164
	WESLEY DR	908 WESLEY DR
	WEST AVE	2064 S WEST AVE
	WOODFIELD	105 CAMBRIDGE AVE

1.2.2 SEWER INVENTORY

The City's collection system consists of approximately 267 miles of gravity sewer ranging from 6 to 60 inches in diameter and approximately 24 miles of force main ranging from 2 to 30 inches in diameter. While newer force mains are generally constructed of PVC, approximately 11 miles of ferrous force mains still exist.

Table 2 – Gravity Sewer Inventory

Sum of Length	Colum										
Diameter	**AC**	**CI**	**CL**	**CON**	**DCI**	**HDPE**	**PVC**	**RCON**	**UNK**	**Unknown**	**Grand Total**
Unknown			0.10							0.02	0.12
8	4.75		20.27	61.87		0.21	119.74	0.16	0.72	4.35	212.06
10	0.21		2.62	5.61			5.78			0.81	15.03
12	0.20	0.12	0.92	0.68			6.25	0.58		0.01	8.77
14	0.20				0.03		0.06				0.28
15		0.02	0.95	2.86			3.40	3.27			10.50
16	0.26	0.05					0.52				0.83
18			0.10	0.63			0.60	2.57		0.07	3.96
20	0.07		0.22								0.30
21				0.20			0.80	2.94			3.94
24	0.21	0.03	0.29	0.31			0.24	3.07		0.09	4.23
27							0.17	1.13			1.31
30					0.03		0.11	1.97			2.11
36			0.06	0.28			0.28	1.67		0.37	2.66
42							0.11	0.55			0.66
48							0.03	0.03		0.22	0.28
54							0.06	0.03			0.08
60							0.04				0.04
(blank)											
Grand Total	**5.90**	**0.21**	**25.53**	**72.45**	**0.06**	**0.21**	**138.30**	**17.97**	**0.72**	**5.99**	**267.33**

A complete inventory of force mains has been provided in Section 7.1.

1.2.3 SEWER CONDITION / AGE

Some of the oldest sewers in the City are over 100 years old. While age information is not available for 23% of the sewers, these are predominantly around the perimeter of the service area, meaning they are likely relatively new. However, some are in the downtown area and are probably older sewers. Figure 6 indicates distribution of pipe with known ages. 75% of the system is less than 50 years old; however, the remaining 25% may be reaching the end of its useful life and warrant physical inspection.

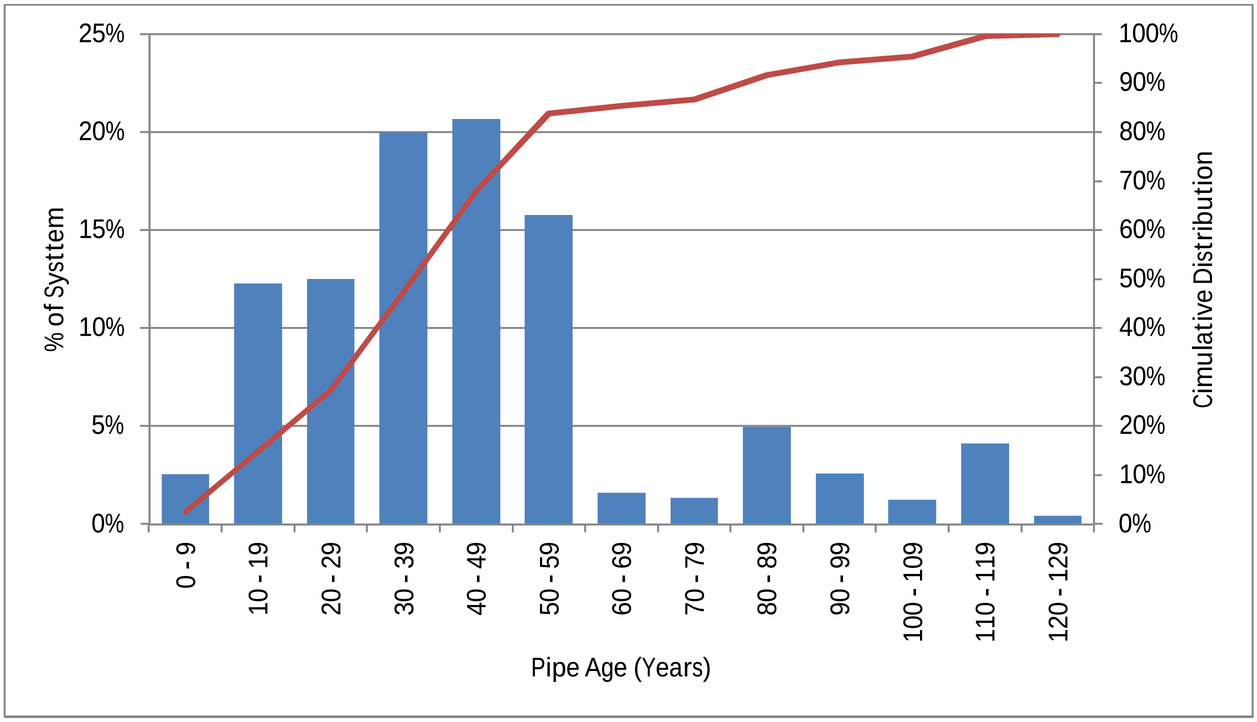


Figure 6 – Sewer Age (Graph)

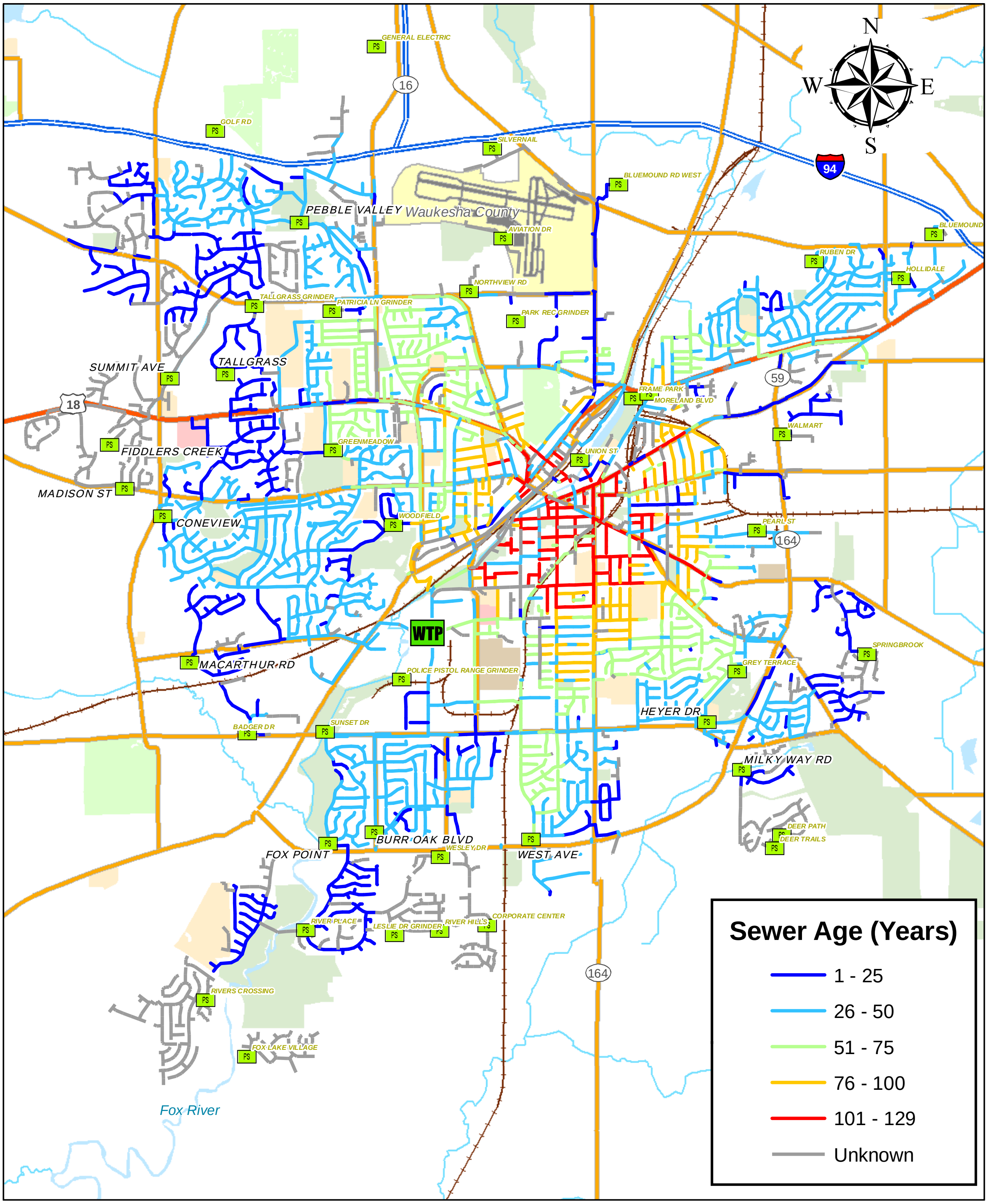


Figure 7 – Sewer Age (Map)

1.3 SYSTEM DEFICIENCIES

Prior to initiating this project, there were several known deficiencies in the collection system as indicated in Figure 8. The City has been working to alleviate these problems in a logical, coordinated manner. For example, storm and sanitary improvements are under development to alleviate flooding along Grandview Avenue. (See below.)

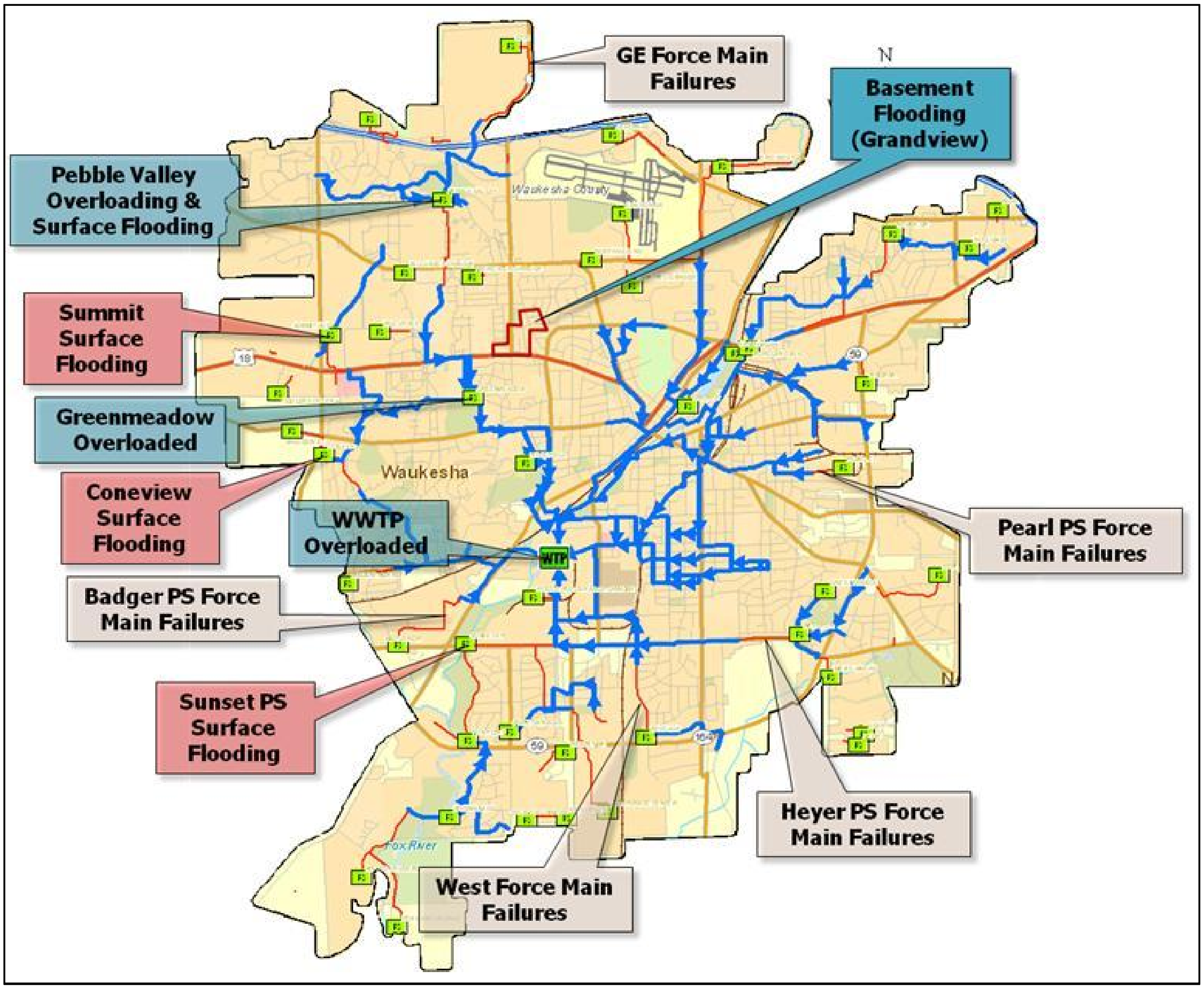


Figure 8 – Known Problem Areas

1.3.1 PEBBLE VALLEY & GREENMEADOW PUMP STATIONS

The Pebble Valley and Greenmeadow pump stations in the northwest part of the City serve populations of 4,500 and 9,300 respectively, making these two of the more critical stations in the collection system. These stations are connected in series, with the Pebble Valley station just upstream of the Greenmeadow station. Wet weather flows can exceed Pebble Valley's capacity. Wet weather flows may also have a cascading effect on the Greenmeadow station since it is receiving wet weather flows from both the Pebble Valley service area and its own. Identifying means to either eliminate these stations and/or significantly reduce clear water flows is vital to providing reliable service to the neighborhoods these stations serve.

1.3.2 Grandview Avenue

In June 2008, a particularly large storm resulted in surface and basement flooding in this area. A questionnaire was mailed out to all the residents in the affected area with 54 returned. These indicated that flooding was caused by both stormwater and sanitary sewer backup. This problem has been alleviated by replacing the Grandview Blvd and Summit Avenue sewers and improved stormwater management.

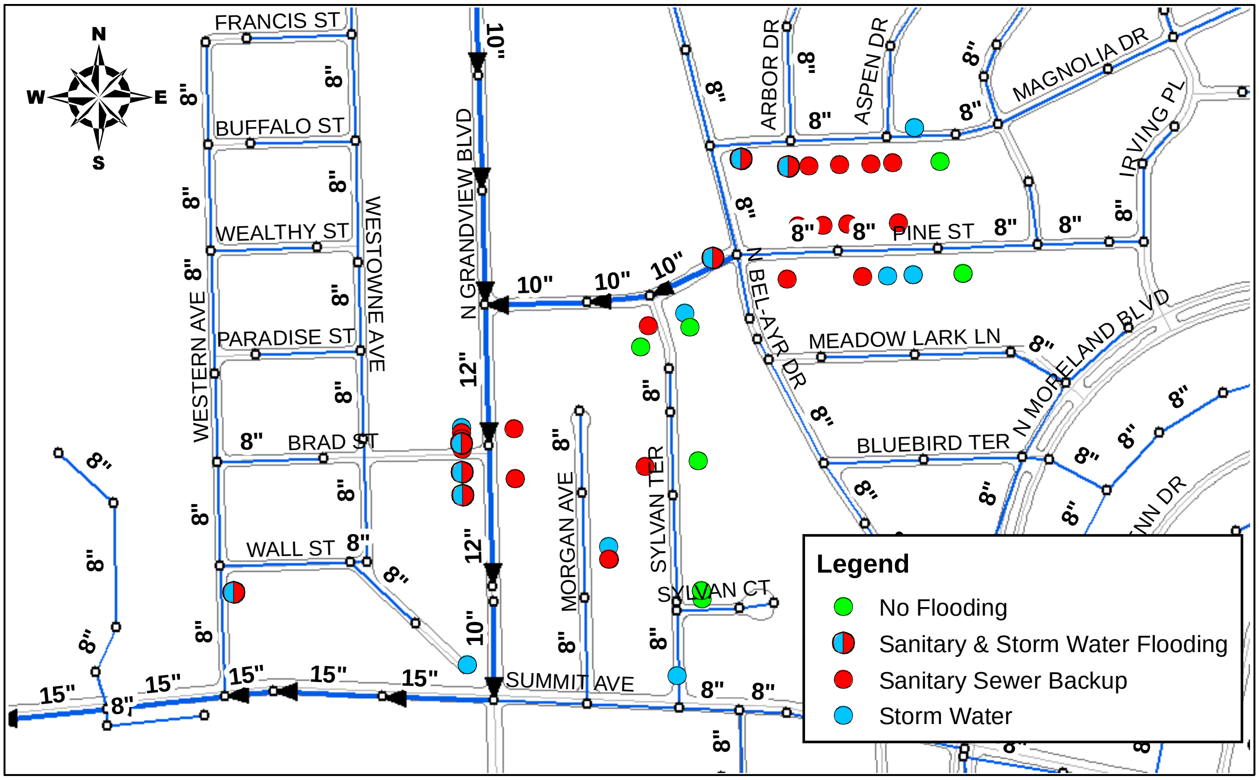


Figure 9 – 2008 Grandview Flooding

1.3.3 Pump Station Surface Flooding

Several pump stations are located in areas prone to surface flooding from storm runoff, particularly those in low-lying areas. Crews have had to sandbag around the Pebble Valley, Sunset, and West Ave pump stations; water has been over the hatches at Summit from the adjacent creek and at Coneview from the adjacent detention basin. During the storm of June 2008, the Summit pump station was inundated and lost power. Chapter II of this report discusses flood prone stations and potential flood protection measures.

1.4 Inflow & Infiltration

As with any sanitary sewer system, the City's collection system is prone to the intrusion of clear water flows in the form of inflow and infiltration (I&I). The nature and severity of the intrusion varies greatly throughout the service area, with the older parts of the system likely contributing the majority of I&I. While the collection system is able to convey all but the largest of storm events, it can present challenges in certain portions of the system—Pebble Valley and Greenmeadow for example. This is discussed in greater detail in Chapter 4.

CHAPTER II –PUMP STATION FLOOD PROTECTION

2.1 INTRODUCTION

During large storm events, the Aviation Drive, Coneview, Pebble Valley, Summit Avenue, and Sunset Drive sanitary pump stations are at the greatest risk from flooding from stormwater runoff. The Sunset Drive, Fox Point, Badger Dr, Union St, MacArthur Road, West Ave, and Frame Park stations are within/adjacent to the 100-year floodplain. Appendix A contains detailed site plans for each of the recommendations in this chapter.

In addition to the recommendations contained in this chapter, the City will consider other flood protection alternatives. For example, the City has already decided that rather than implement the modifications recommended herein, they are going to raise the deck and control panels out of the flood plain.

Figure 10 – Pump Stations within the 100-Year Floodplain

2.2 AVIATION DRIVE

This pump station sits in a local depression slightly below El 904. Surface drainage from the largely impervious area immediately surrounding the pump station ponds around the station and can cause flooding of the station.

Donohue recommends re-grading the area south of the pump station to direct the water away from the station. Soil removed to allow drainage to the south could be deposited around the station to direct surface runoff away from the station. Additionally, a catch basin could be installed near the station with piping to drain to the southwest where it could be discharged below El 900.

2.3 CONEVIEW

Surface runoff flows along the access drive toward the pump station. Overflow from the detention basin to the east may also flow toward the pump station.

Donohue recommends re-gradng the access drive to place a "hump" in it that will shed the water flowing down the drive toward the station to either side of the drive where re-graded swales will direct the water around the station. To assist with directing water around the station, landscaping timbers (railroad ties) anchored to the ground securely (to prevent water from moving them) at least 16 inches high should be placed along the edge of the drive/parking area to further protect the station. The overflow from the detention basin should also be re-graded to direct the water away from the station.

2.4 PEBBLE VALLEY

High water in the adjoining wetland can flood the pump station.

Donohue recommends constructing a 3-foot high earth levee around the pump station, parking area, and electrical transformer. The levee should be constructed of clay and silt materials or other low permeability soils. The levee slopes would be approximately 4H:1V to make them easily mowable. The levee should have an 8 to 10-foot wide flat top cross section. Across the entrance driveway, the slope would be flattened to approximately 10H:1V to allow vehicular access to the station area. Re-grading the driveway area would require removing all the driveway paving and replacing the proposed grades. Due to the limited space between the station and Pebble Valley Road, the re-grading would have to continue all the way to the back of the curb on Pebble Valley Road and would also require re-grading some sidewalk areas.

The interior area (approximately 115 feet by 85 feet, 0.22 acres) would capture rainwater. In order to provide a stormwater outlet, an inlet in front of the pump station connected by a 12-inch gravity connection to the storm manhole on the southwest corner of the site should be made. To prevent backwater from the wetland from entering the interior of the protected area through that connection, a tideflex-style check valve should be installed on the connection. On the rare occasions when high rain and high water in the wetland occur simultaneously, an emergency connection into the pump station influent may be necessary to drain the interior of the levee area. Although this emergency connection will introduce stormwater into the sanitary system it would be less costly than constructing a separate stormwater pumping system for these infrequent events and overall stormwater into the pump station will be greatly reduced.

2.5 SUMMIT AVENUE

High water in the adjacent creek or retention basins can potentially flood the station. While not in the floodplain, the interceptor sewer to the Summit PS parallels a creek bed that may overtop the manholes during large storms. There is a manhole (#4374) adjacent to a detention pond at the northern end of it that might overflow into the pump station. These risks may explain why this sewer experienced unusually high inflow during the June 2008 storm (Figure 11). City staff said hatches at this station have been underwater.

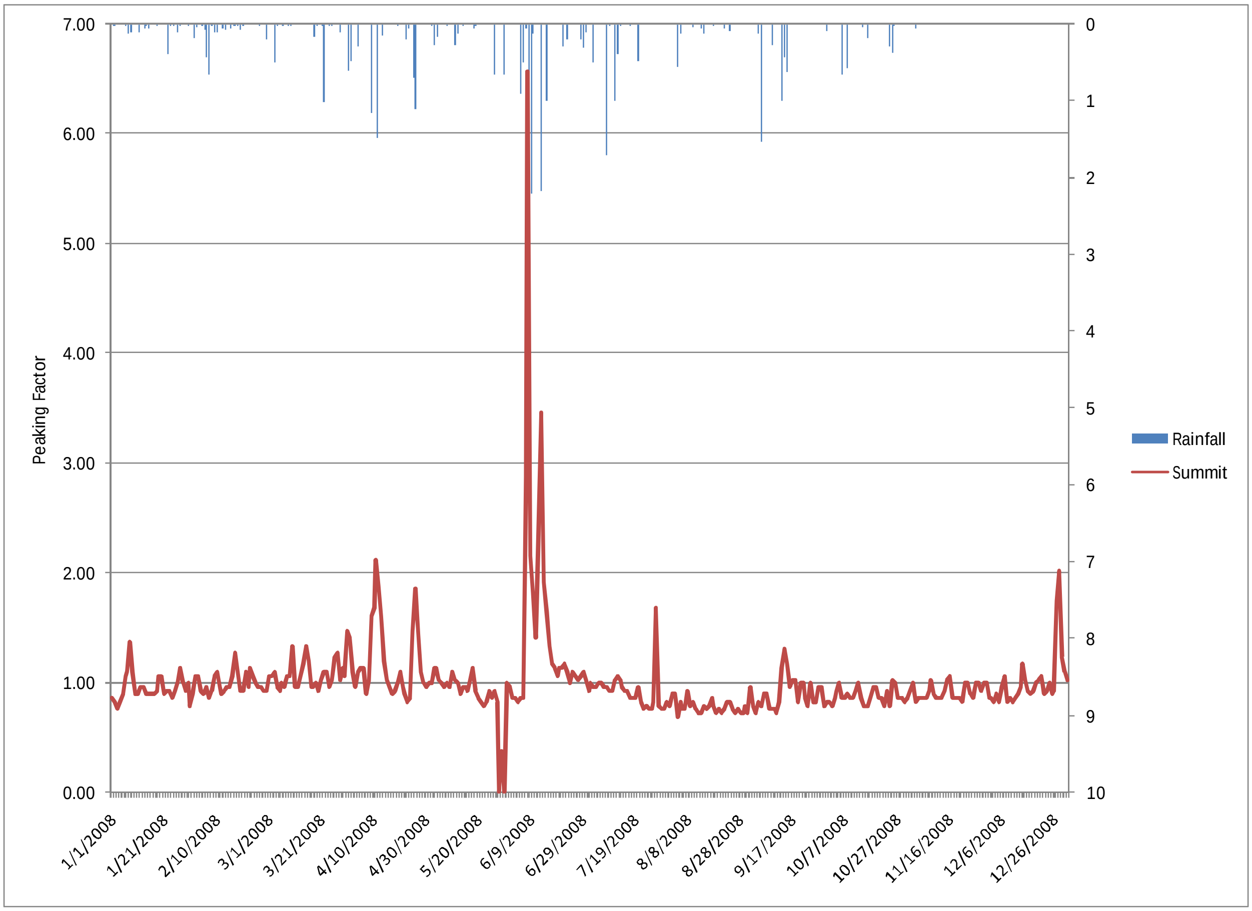


Figure 11 – Summit Avenue Pumping (June 2008 Storm)

Donohue recommends protecting the station with a combination of low earth levees and concrete walls around the pump station, parking area, and electrical facilities (if required). The levee should be constructed of clay and silt materials or other low permeability soils. The levee slopes should be approximately 4H:1V or flatter to make them easily mowable. The levee should have an 8-foot wide flat top. Re-grading of the parking area to accommodate the levee grades would be flattened to approximately 10H:1V for vehicular access. Re-grading the driveway area would require removal of all the driveway paving and replacement at the proposed grades. Due to the limited space between the station and adjoining creek and wetland areas, a concrete wall approximately 4 feet high would be needed on the South and West sides with a short piece on the north side in order to tie into the levee section. Installing the levee on the eastern and northern sides of the pump station parking area would ensure that high water does not go around the ends of the concrete walls and that overflows from the pond area would be directed around the station.

In order to drain the interior area (approximately 65 feet by 65 feet, 0.10 acres), a 12-inch stormwater outlet pipe should be installed in front of the pump station at the low point in the parking area to discharge into the creek. To prevent backwater from the creek from entering the interior of the protected area, a tideflex-style check valve should be installed on the connection. On the rare occasions when heavy rainfall and high water in the creek occur simultaneously, an emergency connection into the pump station influent would keep the interior of the area dewatered. Although this emergency connection will introduce stormwater into the sanitary system, it would be less costly than constructing a separate stormwater pumping system for these infrequent events and overall stormwater into the pump station will be greatly reduced.

2.6 SUNSET DRIVE

High water (flood elevation 795) in the adjoining river can potentially flood the pump station, which lies about 8 inches below the 100-year flood elevation. During the June 2008 storm, this station pumped an unusually high volume of water for a prolonged period of time. This is a strong indicator that it was pumping surface water entering the station through the hatches.

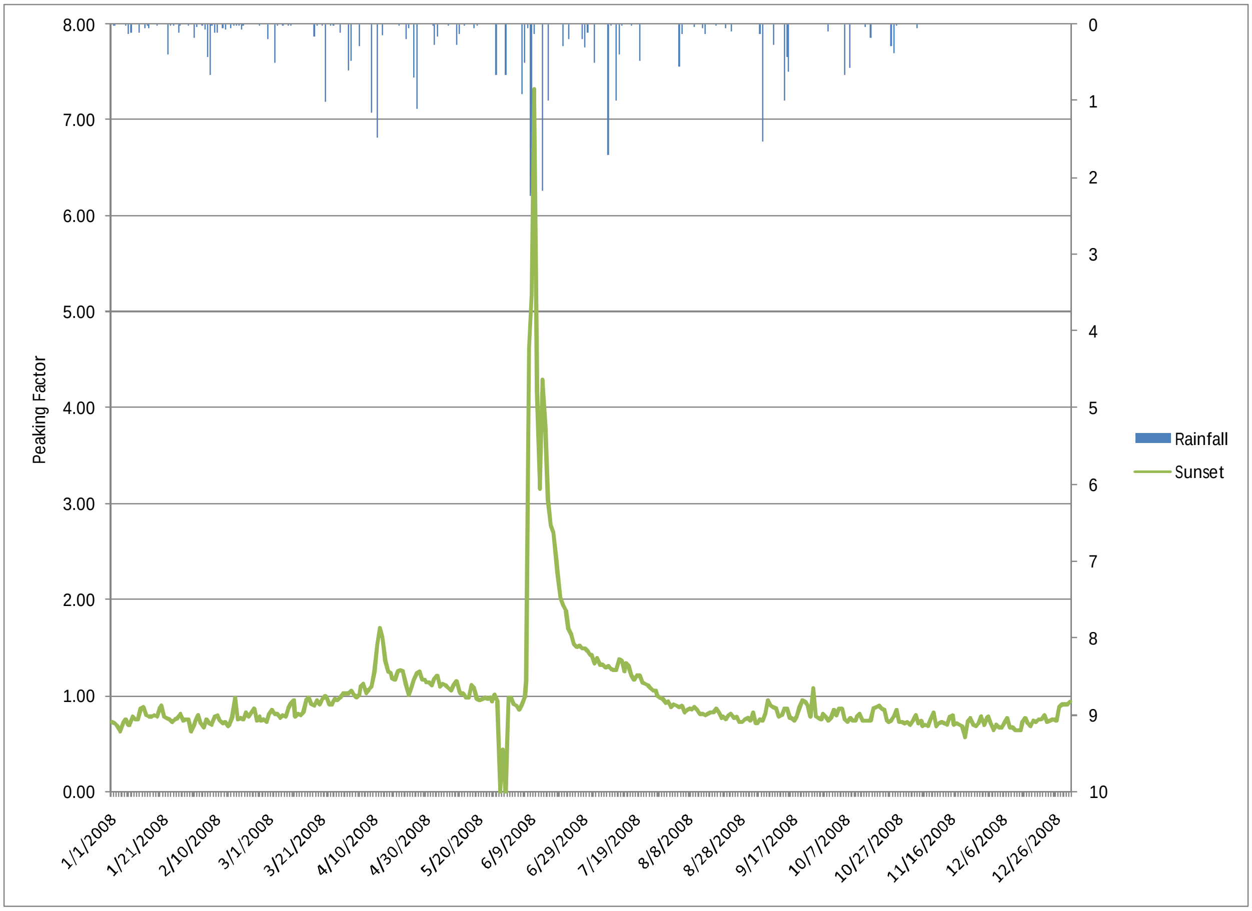


Figure 12 – Sunset Drive Pumping (June 2008 Storm)

Donohue recommends protecting the station with a combination of low earth levees and a concrete wall around the pump station and parking area to elevation 798 (3 feet above flood elevation per urban standards). The levee should be constructed of clay and silt materials or other low permeability soils. The levee slopes should be approximately 4H:1V to make them easily mowable. The levee should have a flat top cross section that would be about 5 to 8 feet wide (wider is preferable, but the available property is limited). Across the entrance driveway, the slope should be flattened to approximately 10H:1V to allow easy vehicle access into the station area. Re-grading the driveway and parking area would require removal of all the driveway and parking lot paving and replacement at the proposed grades. Due to the limited space between the station and the western property line, rapidly falling grades in that vicinity, and existing wetlands adjoining the property, a concrete wall approximately 4 feet high will likely be needed along the western edge of the parking area between the Sunset Drive right of way and the rear property line. Due to the limited space between the station and Sunset Drive, re-grading work would have to extend all the way to the back of the curb on Sunset Drive in order to achieve the desired slopes and may also require re-grading some sidewalk areas.

The interior area (approximately 90 feet by 90 feet, 0.18 acre) would capture rainwater and not provide an outlet for it. In order to provide a stormwater outlet, a stormwater inlet should be constructed in front of the pump station connected by a 12-inch gravity connection to a storm manhole (near the Sunset Drive sidewalk and driveway entrance), and then discharging near the existing storm outfall approximately 30 feet west of the west property line. Since backwater is possible at that location, a tideflex-style check valve in the manhole at the driveway should be installed on the connection. On the rare occasions when high water in the river closes the outlet and heavy rain occur simultaneously, an emergency connection into the pump station influent may be necessary to keep the interior of the levee area dewatered. Because the capacity of this station is only about 2 times its estimated peak flow (although the peak flows would be less than 5 minutes in duration) it may be prudent to install an interior stormwater pumping system. We recommend performing a risk analysis to assess the impact of introducing infrequent stormwater flows into the sanitary system.

2.7 COST ESTIMATES

The probable construction costs for flood protection at the five pump stations are summarized below in Table 3. Detailed Cost Estimates are attached as Appendix A.

Table 3 – Pump Station Flood Protection Cost Summary

Pump Station	Probable Construction Cost
Aviation Drive	$26,000
Coneview	$36,700
Pebble Valley	$55,600
Summit	$64,000
Sunset	$66,100
Total	$248,400

CHAPTER III – PUMP STATION ELIMINATION

3.1 WEST-SIDE BYPASS

A gravity bypass sewer could potentially eliminate the following seven pump stations: Pebble Valley, Coneview, Heritage Hills, Fiddler's Creek, Summit, Tallgrass, and MacArthur Road. This sewer would be designed to match the capacities of the stations it replaces. Two possible routes are being considered for this bypass as indicated in Figure 13. Profiles of both routes have been included as Figure 14 through Figure 17. Note that the alternate route would be extremely deep, and would therefore require significant lengths of trenchless pipe installation. The design flow from Tallgrass was estimated and the pipe size was chosen to match the size of the existing influent pipe to the Tallgrass wet well.

This bypass could eliminate the Pebble Valley pump station, which is prone to surface flooding and overloading during wet weather events. It would also offload a significant portion of what must currently be pumped by the Greenmeadow pump station, which is at risk of overloading during large rainfall events, and would eliminate the Coneview and Summit pump stations, which are prone to surface flooding.

It would cost approximately $6M to remove six of the seven pump stations. Due to its remote proximity, the cost to extend this sewer to collect flows from Pebble Valley increases the total cost by $5M. Table 5 is a more detailed opinion of probable costs. Costs for the alternate route have yet to be developed as it would require significant use of trenchless construction methods; these unit costs tend to be very localized and have not yet been provided.

Table 4 – West-Side Bypass Cost Estimates

Pump Station(s) Eliminated	Cost Estimate
Coneview, Heritage Hills, Fiddler's Creek, Summit, Tallgrass, MacArthur Road*	$6,000,000
Pebble Valley	$5,000,000
Total	$11,000,000
Energy and O&M Savings	TBD

*Depends on selected route.

3.2 SOUTHEAST BYPASS

A second gravity sewer under consideration is the Southeast Bypass. This gravity sewer would eliminate the following four pump stations: Heyer Drive, West Avenue, Milky Way Road, and Burr Oak Blvd. The flow from these stations would be consolidated at what is currently the Fox Point pump station. This station and force main would have to be replaced to accommodate the additional flow. The proposed sewer route is indicated in Figure 18. Eliminating the Heyer Drive and West Ave force pump stations would eliminate the need for what have been two of the more problematic force mains. The estimated cost to design and construct this bypass sewer is $6.85M. A detailed estimate of probable cost has been included (Table 6). This does not include the cost to replace the Fox Point pump station and force main.

Figure 13 – West-Side Bypass Sewer

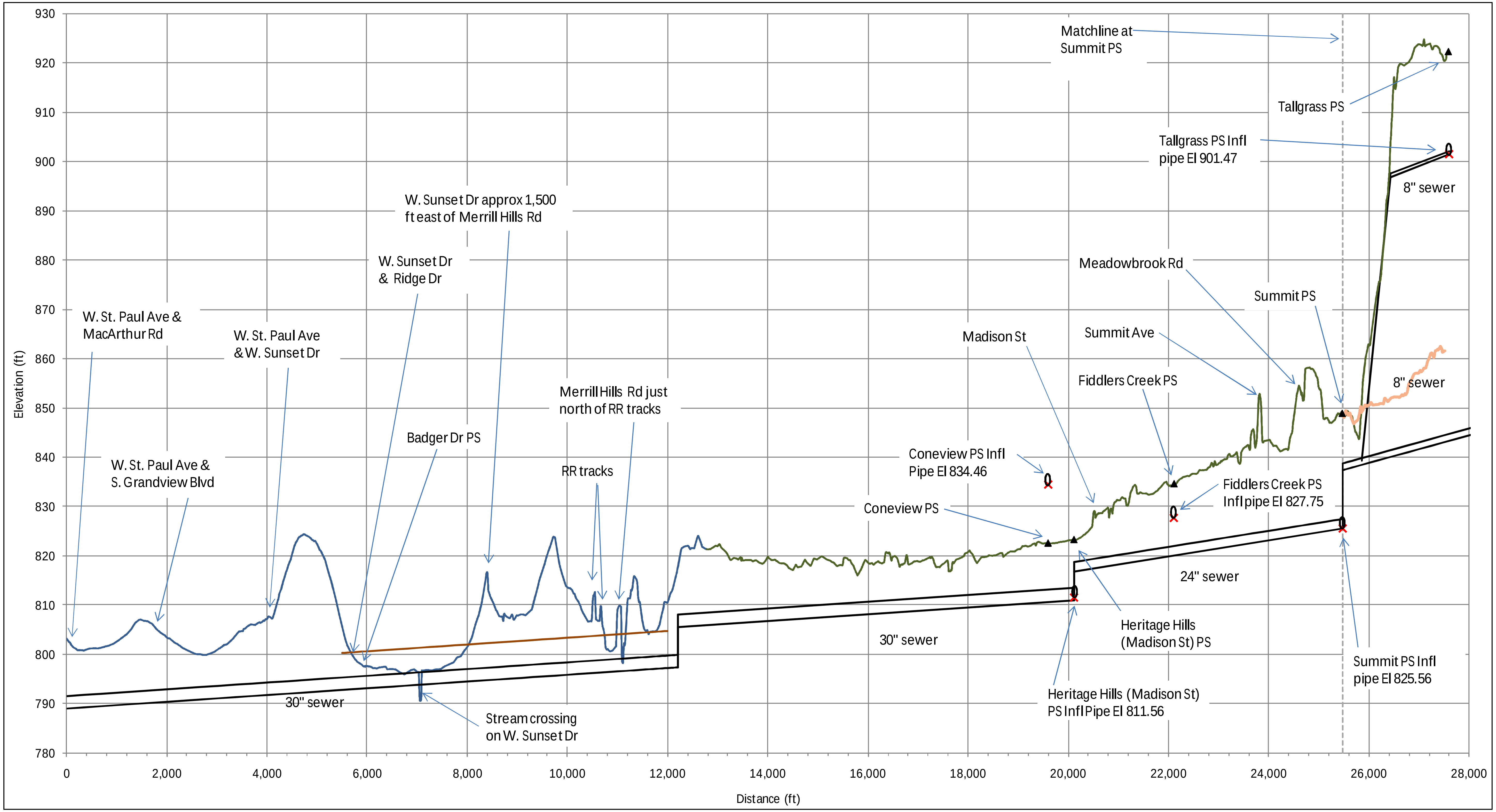


Figure 14 – West-Side Bypass Profile Part I

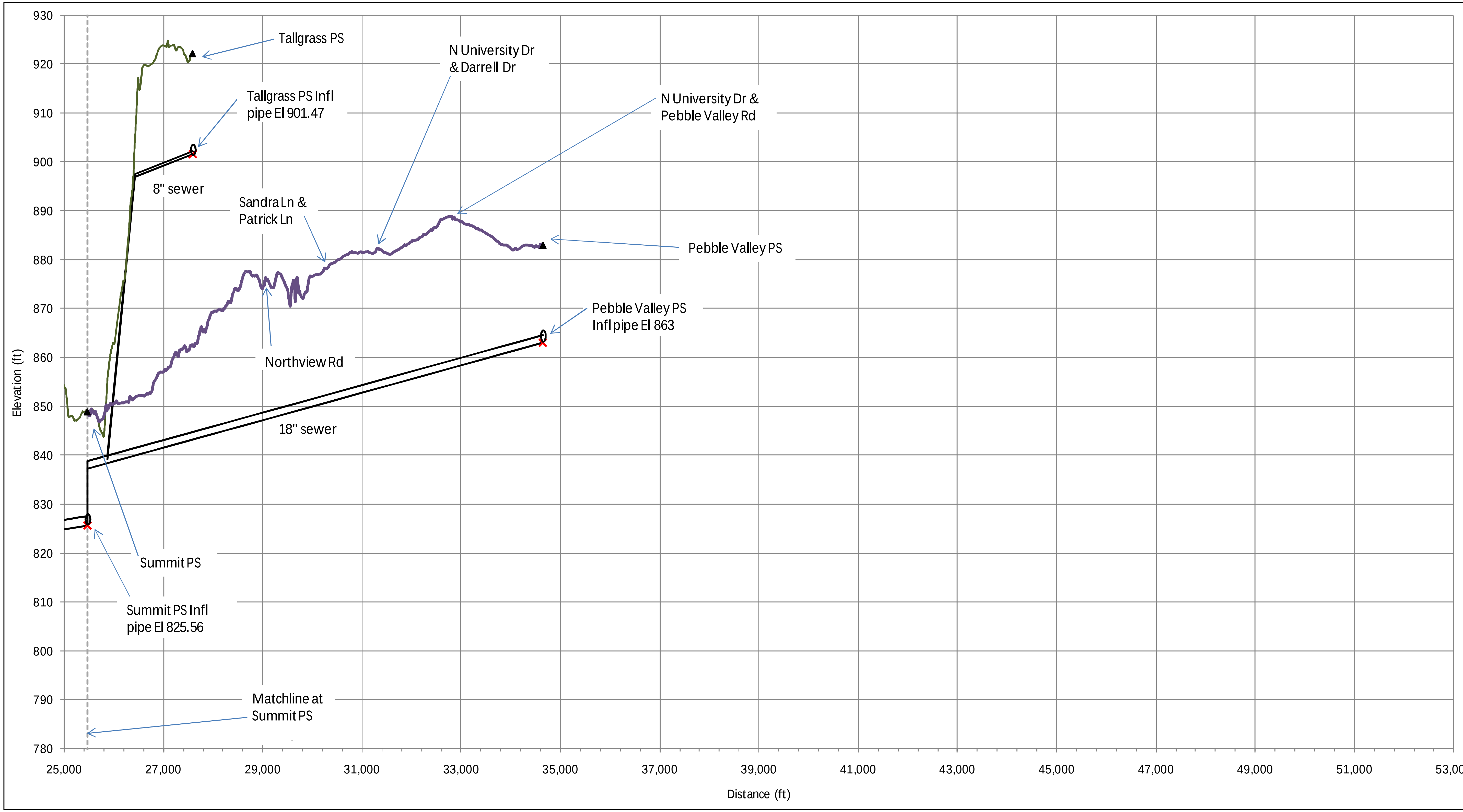


Figure 15 – West-Side Bypass Profile Part II

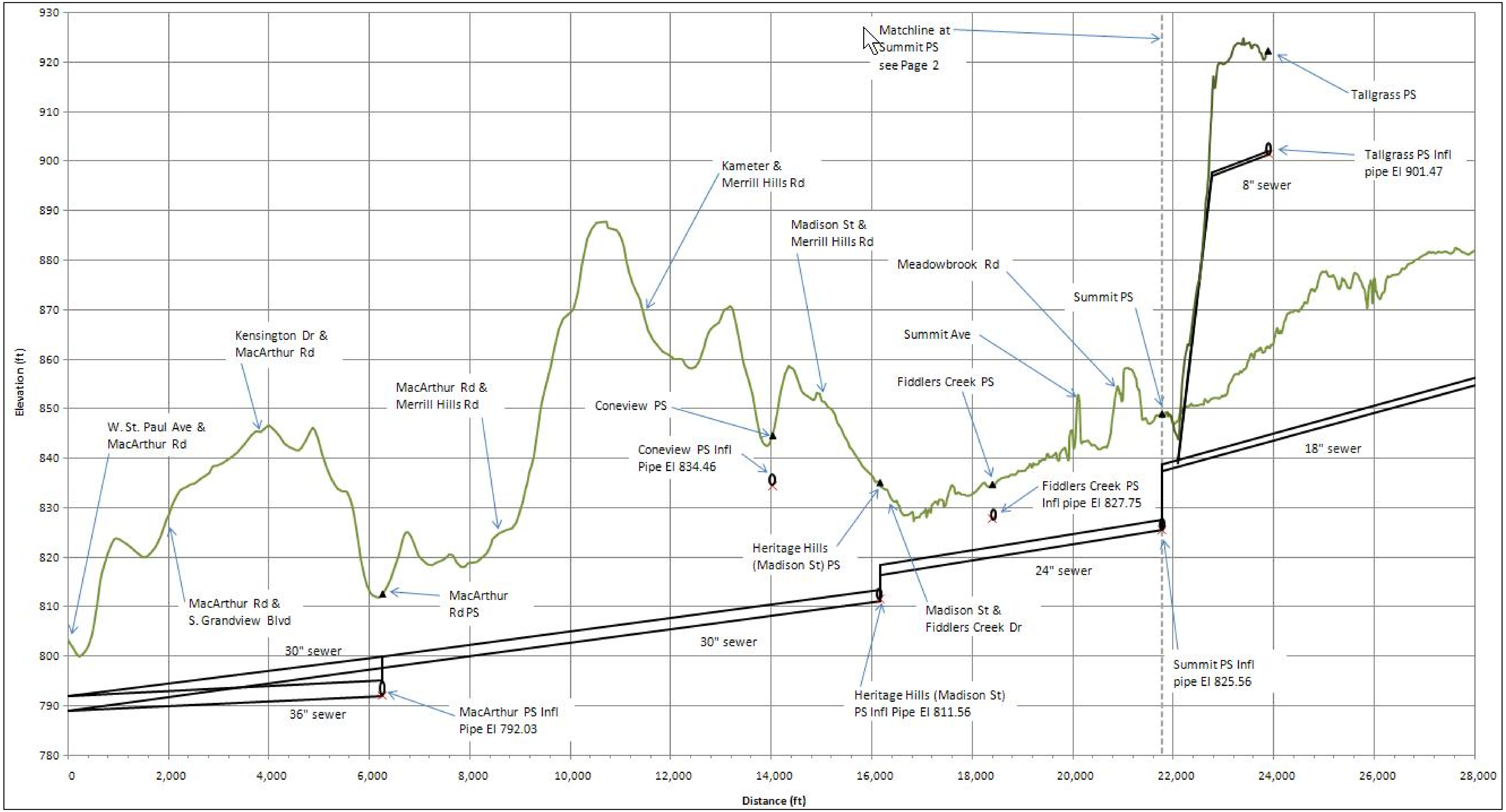


Figure 16 – West-Side Bypass Profile Part I (Alternate)

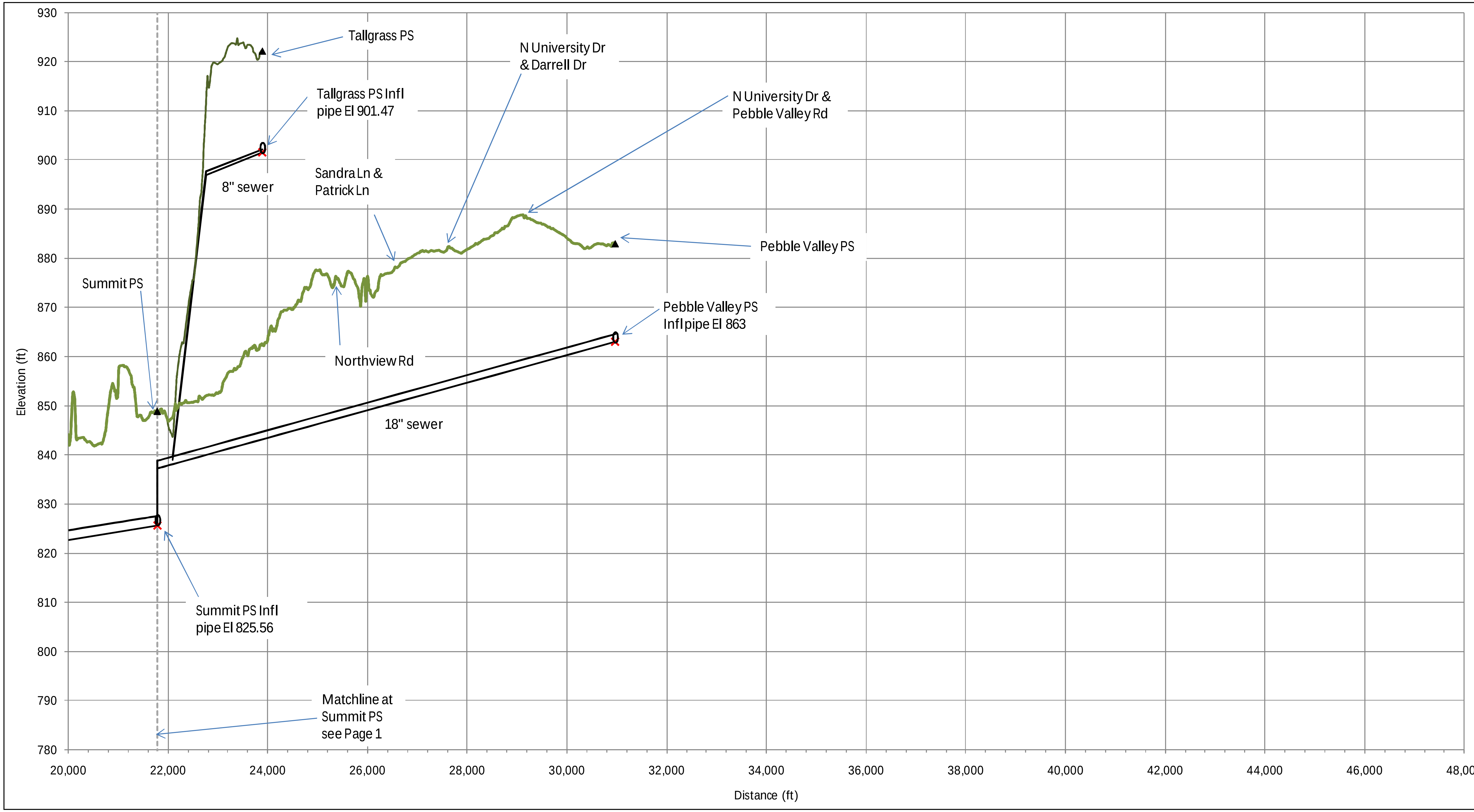


Figure 17 – West-Side Bypass Profile Part II (Alternate)

Table 5 – West-Side Bypass Sewer Cost Opinion

				Length of each type in segment				Cost of each type by depth						
Pipe Segment[1]	Length (ft)	Dia (in)	Depth (ft)	Open (ft)	Semi-congested (ft)	Bore (ft)	Semi-congested road (ft)	Open $/L.F.	Semi-congested $/L.F.	Bore $/L.F.	Semi-congested road $/L.F.	# Manholes[2]	Manhole[3] $/EA	Segment Cost
1	4,000	30	12.7	0	0	0	4,000				343	8	3,764	$1,402,000
2	1,500	30	24.8	1,500	0	0	0	345				3	7,456	$540,000
3	2,700	30	4.6	2,700	0	0	0	155				5	1,429	$426,000
4	1,800	30	17.3	1,800	0	0	0	254				4	5,139	$478,000
5	2,200	30	11.7	2,200	0	0	0	205				4	3,469	$465,000
6	1,800	30	14.4	1,800	0	0	0	195				4	4,268	$368,000
7	6,100	30	10.8	5,600	500	0	0	158	181			12	3,206	$1,014,000
8	4,300	24	15.3	2,800	1,500	0	0	98	121			9	4,536	$497,000
9	1,070	24	26.8	1,070	0	0	0	146				2	8,090	$172,000
10	570	8	6	350	220	0	0	46	69			1	1,824	$33,000
11	1,163	8	21.9	753	410	0	0	94	111			2	6,549	$129,000
12	9,181	18	23.6	3,470	1,000	0	4,711	116	139		329	18	7,079	$2,219,000
13	1,500	21	6	1,500	0	0	0	150				3	1,824	$230,000
Total L =	37,884			25,543	3,630	0	8,711							

Subtotal	$7,973,000
20% Contingency	$1,594,600
15% Engineering	$1,435,000
Total	$11,002,600

[1] Pipe Segments shown and labeled by number on Route Map
[2] # Manholes calculated based on 500 ft spacing
[3] Manhole calculated based on RSMeans 4' diameter manhole and average depth for the segment

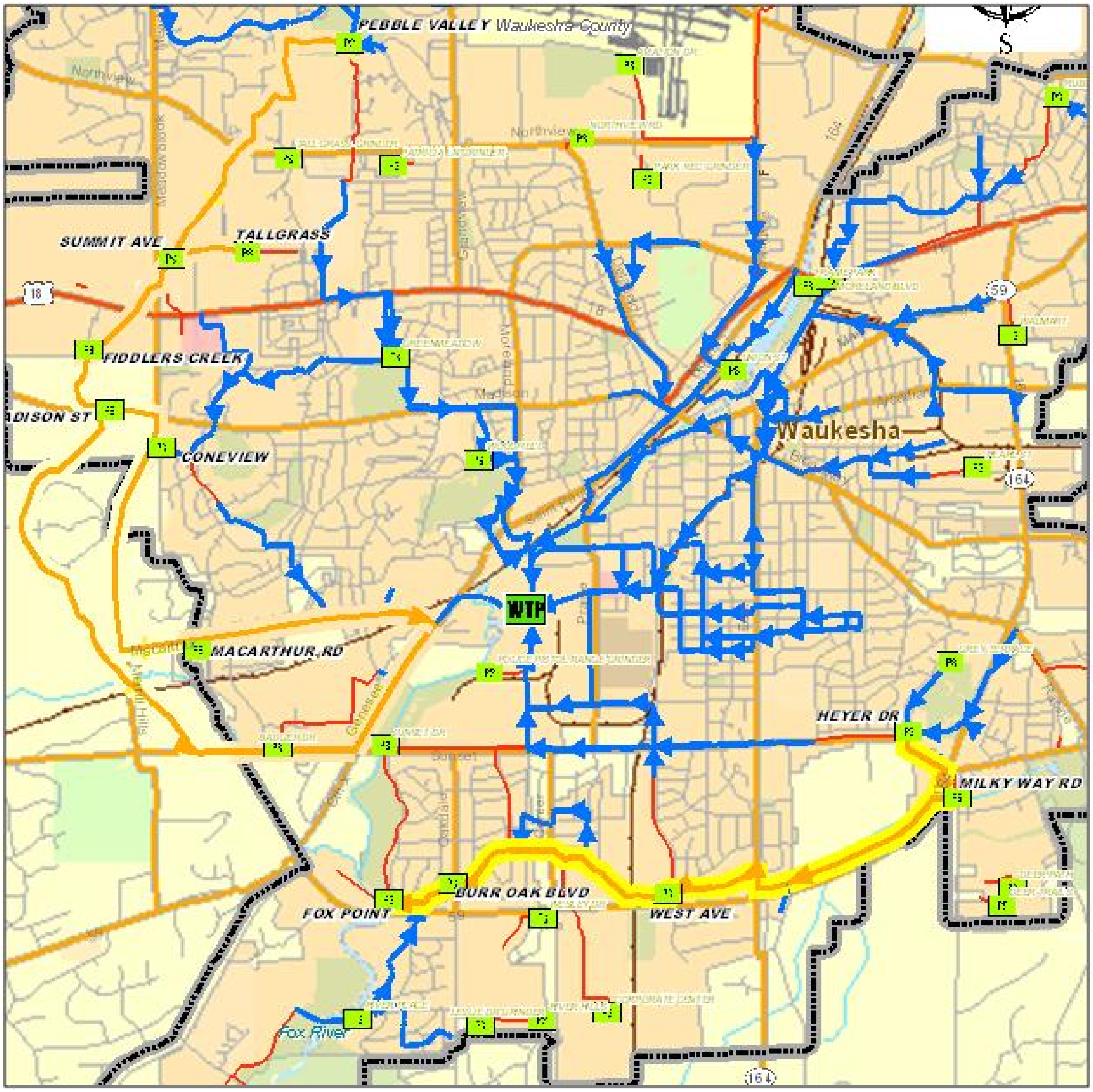


Figure 18 – Southeast Bypass Route

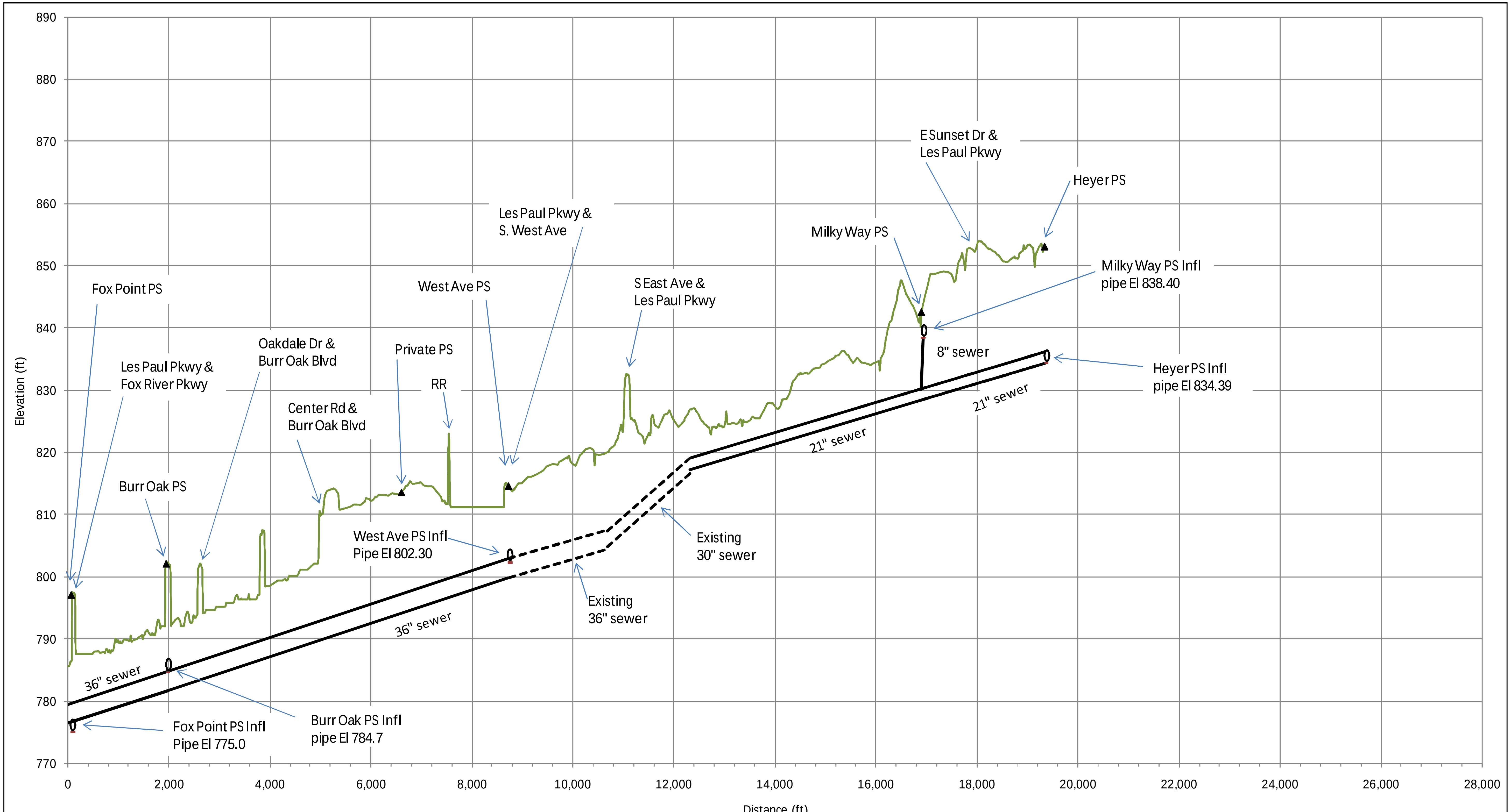


Figure 19 – Southeast Bypass Sewer Profile

Table 6 – Southeast Bypass Sewer Cost Opinion

				Length of each type in segment				Cost of each type by depth						
Pipe Segment[1]	Length (ft)	Dia (in)	Depth (ft)	Open (ft)	Semi-congested (ft)	Bore (ft)	Semi-congested road (ft)	Open $/L.F.	Semi-congested $/L.F.	Bore $/L.F.	Semi-congested road $/L.F.	# Manholes[2]	Manhole[3] $/EA	Segment Cost
1	1,949	36	10.5	1,949	0	0	0	283				4	3,119	$564,000
2	6,776	36	15.1	6,776	0	0	0	312				14	4,477	$2,177,000
3	1,947	36	Existing									N/A	N/A	N/A
4	1,645	30	Existing									N/A	N/A	N/A
5	4,580	21	9.1	4,580	0	0	0	175				9	2,712	$826,000
6	2,441	21	19.4	0	0	0	2,441				536	5	5,778	$1,337,000
7	600	8	8	600	0	0	0	91				1	2,395	$57,000
Total L =	19,938			13,905	0	0	2,441							
													Subtotal	$4,961,000
													20% Contingency	$992,200
													15% Engineering	$893,000
													Total	$6,846,200

[1] Pipe Segments shown and labeled by number on SE PS Elimination Route Map
[2] # Manholes calculated based on 500 ft spacing
[3] Manhole calculated based on RSMeans 4' diameter manhole and average depth for the segment

3.3 MISC. PUMP STATIONS

The City intends to eliminate the River Hills (Dana) and Wesley pump stations in 2010 by constructing gravity sewers across Center Road (Figure 20) and tying in to the existing 8-inch sewers.

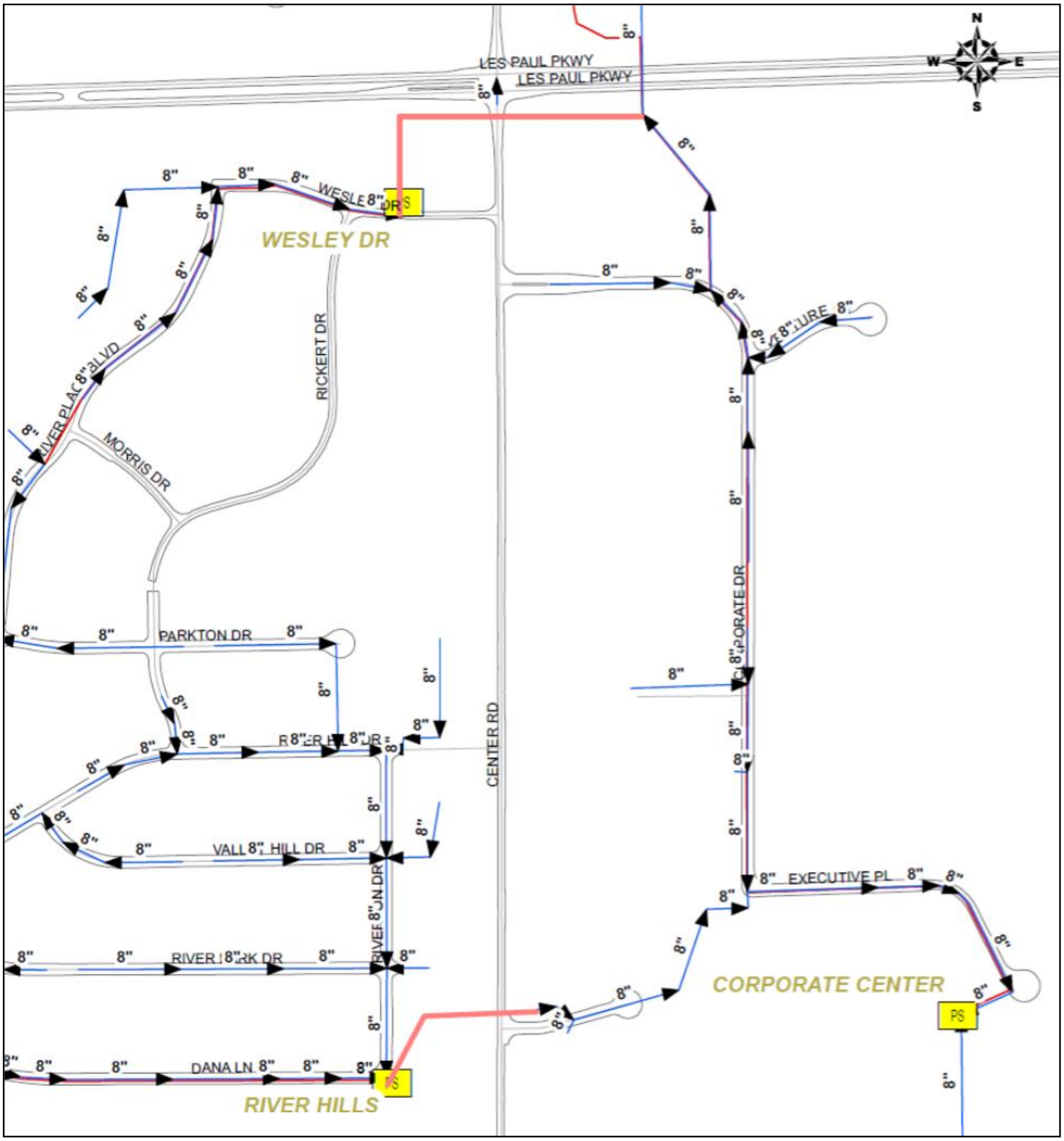


Figure 20 – River Hills & Wesley Dr PS Elimination Routes

Donohue engineers considered how to eliminate the Hollidale Pump Station (Figure 21). This is not recommended as the proposed bypass sewer would need to extend all the way to Ruben Dr Pump Station wet well and would have a wet weather velocity of only 1.4 fps. (See Figure 21 and Figure 22.) The lack of scouring velocities would likely result in sediment deposition.

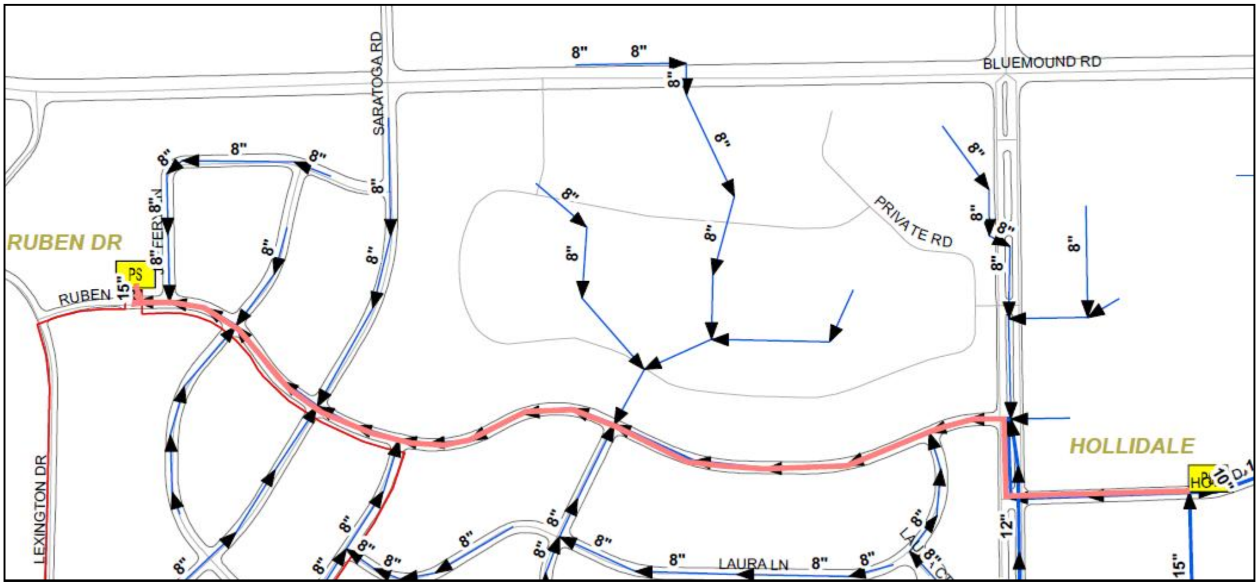


Figure 21 – Hollidale PS Elimination Sewer Route

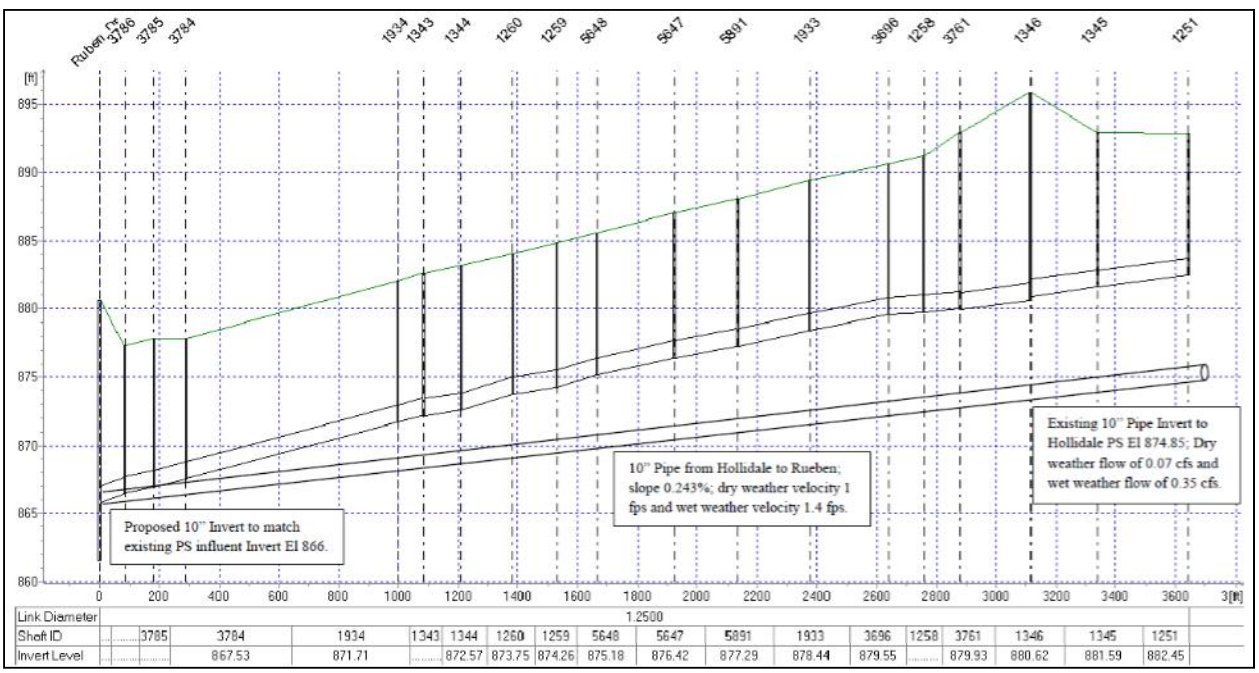


Figure 22 – Hollidale PS Elimination Sewer Profile

Donohue considered the elimination of the Woodfield Pump Station with a gravity sewer. If the City wishes to eliminate this station, Donohue recommends installing an 8-inch sewer that would extend approximately 2,700 ft from Woodfield Pump Station to just south of the intersection of St Paul Avenue and Moreland Boulevard (Figure 23). Alternatively, approximately 1,700 ft of the existing 18" sewer could be lowered by 5-7 feet (Figure 24) to accept Woodfield flow.

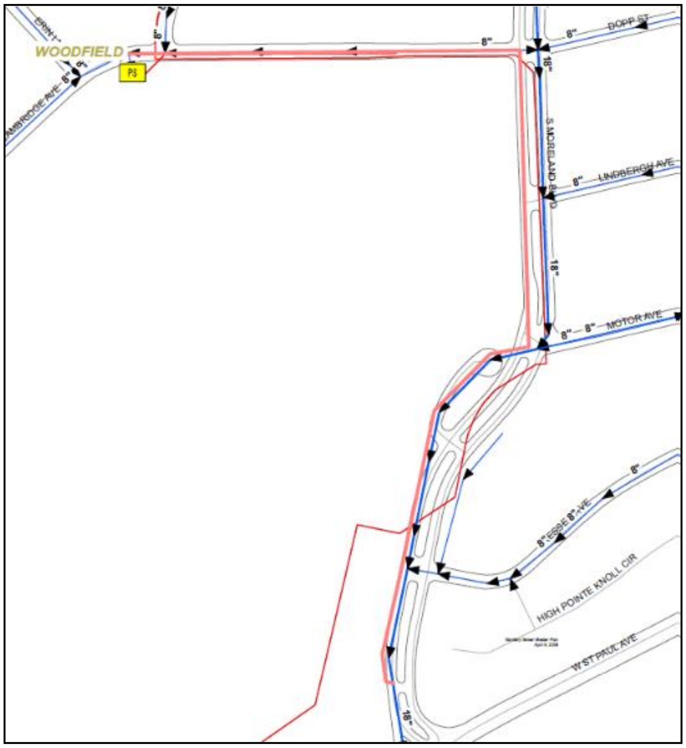


Figure 23 – Woodfield PS Elimination Route

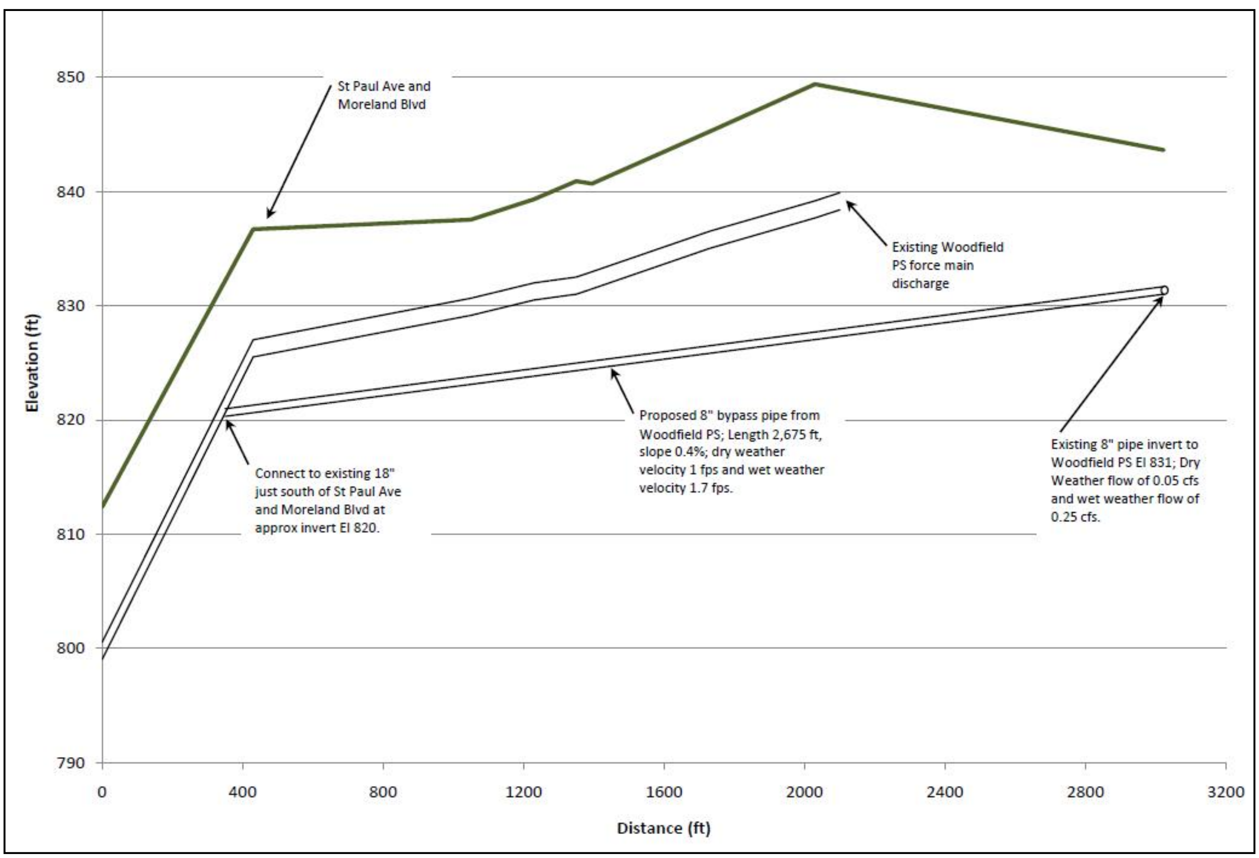


Figure 24 – Woodfield PS Elimination Profile

CHAPTER IV – INFLOW & INFILTRATION EVALUATION

4.1 NATURE OF THE PROBLEM

Inflow and infiltration (I&I) are terms used to describe the ways that groundwater and stormwater enter into dedicated wastewater or sanitary sewer systems. Typical methods of entry are indicated graphically in Figure 26.

Inflow is stormwater that enters into sanitary sewer systems at points of direct connection to the systems. Various sources contribute to the inflow, including footing/foundation drains, roof drains or leaders, downspouts, drains from window wells, outdoor basement stairwells, drains from driveways, groundwater/basement sump pumps, and even streams. These sources are typically improperly or illegally connected to sanitary sewer systems, via either direct connections or discharge into sinks or tubs that are directly connected to the sewer system. An improper connection lets water from sources other than sanitary fixtures and drains to enter the sanitary sewer system. That water should be directed to the stormwater sewer system or allowed to soak into the ground without entering the sanitary sewer system.

Improper connections can be made in either residential homes or businesses and can contribute a significant amount of water to sanitary sewer systems. An 8-inch sanitary sewer can adequately move the domestic wastewater flow from up to 200 homes, but only eight sump pumps operating at full capacity or six homes with directly connected downspouts may overload the capacity of the same eight inch sewer pipes. A single sump pump can contribute over 7,000 gallons of water to sanitary sewer systems in a 24 hour period, the equivalent of the average daily flow from 26 homes.

Infiltration is groundwater that enters sanitary sewer systems through cracks and/or leaks in the sanitary sewer pipes. Cracks or leaks in sanitary sewer pipes or manholes may be caused by age related deterioration, loose joints, poor design, installation or maintenance errors, damage or root infiltration. Groundwater can enter these cracks or leaks wherever sanitary sewer systems lie beneath water tables or the soil above the sewer system becomes saturated. Often sewer pipes are installed adjacent to and/or beneath creeks or streams because they are the lowest point in the area making it less expensive than to install the pipe systems beneath a roadway. These sewer pipes are especially susceptible to infiltration when they crack or break and have been known to drain entire streams into sanitary sewer systems. Average sewer pipes are designed to last about 20-50 years, depending on what type of material is used. Often sanitary sewer system pipes along with the lateral pipes attached to households and businesses have gone much longer without inspection or repair and are likely to be cracked or damaged.

Service laterals can be particularly insidious as these are often poorly constructed and rarely, if ever, inspected until a failure occurs. They are often near trees and shrubs who's roots can penetrate and degrade the lateral. It is not uncommon for service laterals to contribute 50% or more of the total I&I. The municipality often has no jurisdiction over the maintenance of service laterals out of the public right-of-way.

Rainfall dependent infiltration is infiltration that spikes shortly after rainfall events due to increased soil saturation and tapers off slowly over a period of days following the event.

While there are industry-standard metrics by which to quantify inflow and infiltration, and threshold values for what is generally considered excessive, it is important to note that these thresholds are based on a general sense of when it typically becomes more cost-effective to remove I&I than to convey and treat it. However in practice I&I should be evaluated on a case-by-case basis. I&I that enters the sanitary sewer system at a location distant from the treatment plant or into a sewer with limited capacity may present a greater challenge than that entering a sewer near the treatment plant and/or a sewer with excess capacity. Each case must be analyzed and a cost-benefit evaluation comparing the cost to remove vs. the cost to convey and treat developed in order to identify the most cost-effective solution. Donohue will employ a cost-effective evaluation similar to the one portrayed below in order to optimize I&I reduction costs with transport & treat costs.

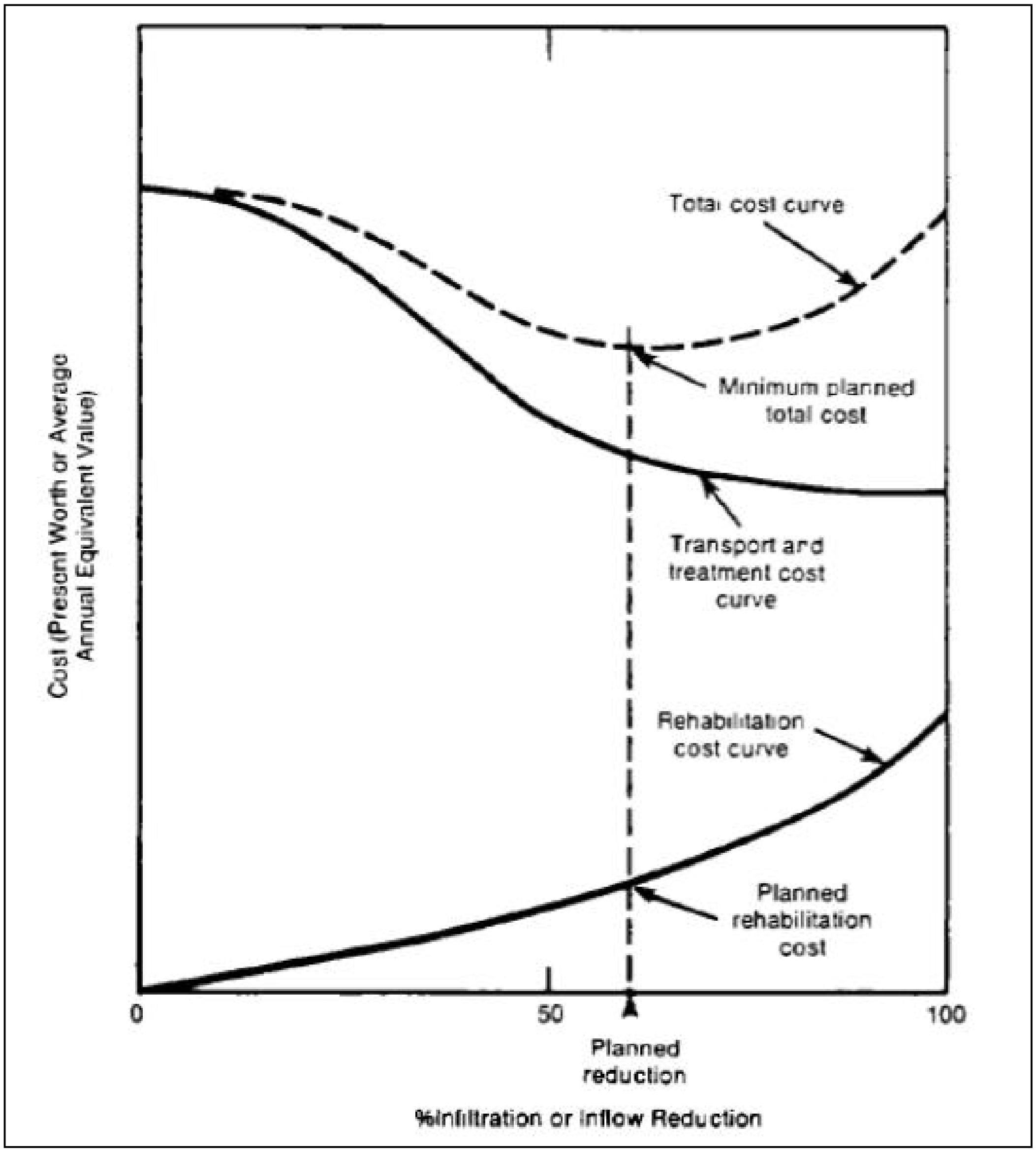


Figure 25 - I&I Reduction Cost-Effectiveness Analysis
(U.S. EPA, 1985)

(City of Bryan, Texas, 2010)

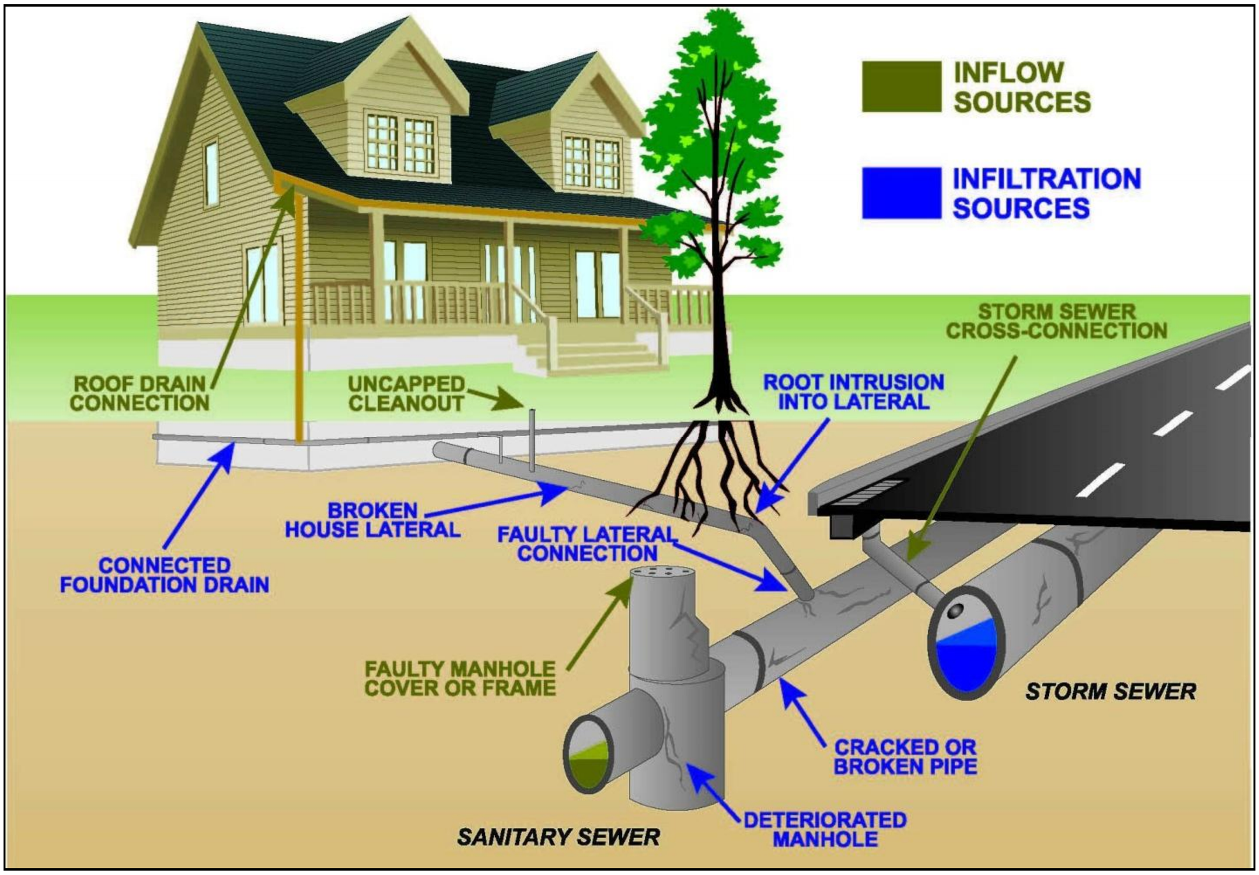


Figure 26 – Typical Sources of Inflow and Infiltration

4.2 System-Wide I&I Quantification

Using four years of flow data (2000 – 2003) from the City water plant and wastewater treatment plant, a mass balance was performed in order to estimate total I&I. This analysis only provides insight as to the total volume of I&I on annual and seasonal bases. It does not distinguish between inflow and infiltration, nor does it specify from which areas these flows are originating.

For this evaluation, summer water supply and consumption data were excluded since a significant portion of the supply is used for irrigation and can distort the results. Over the 4-year period the fraction of flow treated at the WWTP that was I&I ranged from 24% in January to 50% in May with an overall average of 38% (see Figure 27 and Figure 28).

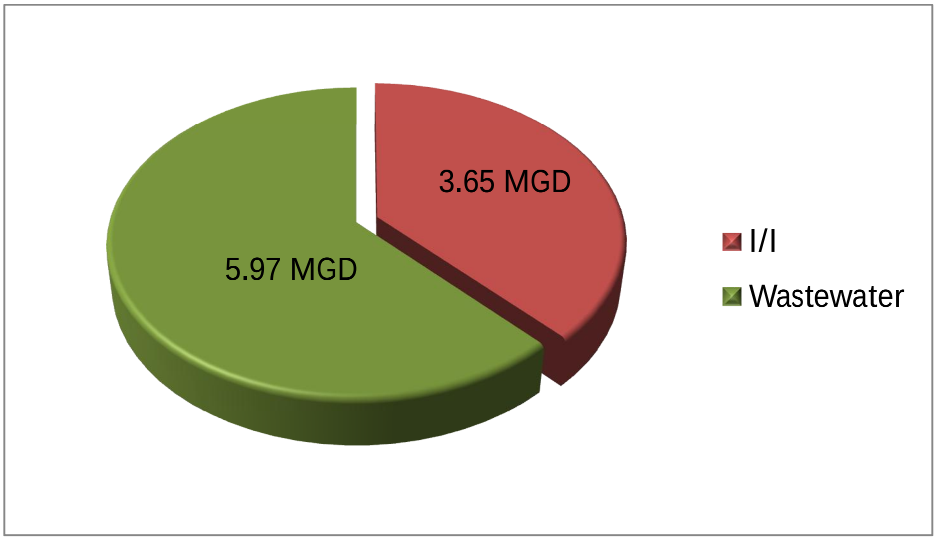


Figure 27 – WWTP Flow Components

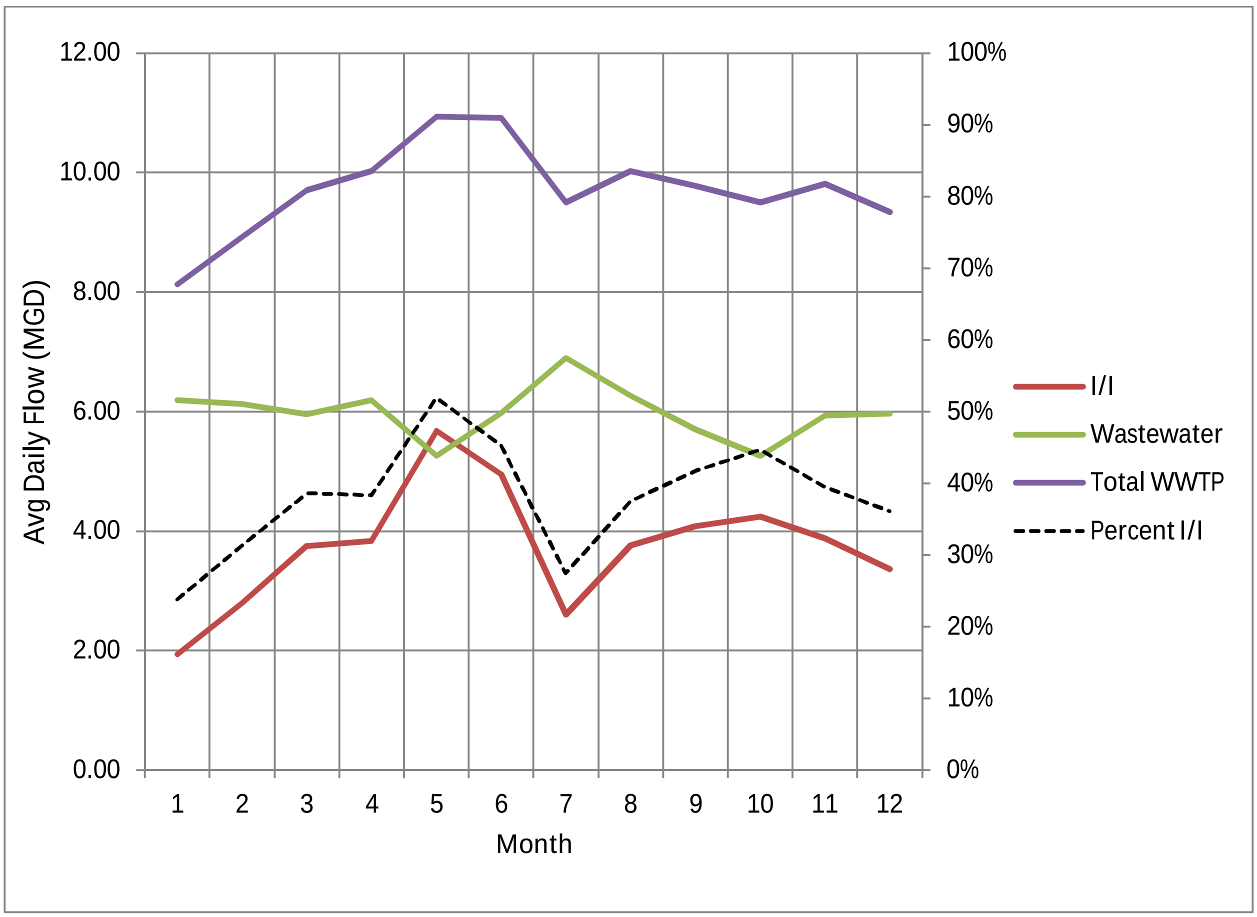


Figure 28 – 2000-2003 Daily Average WWTP Flows

4.3 2008 Pump Run Time Evaluation

The City's extensive network of sanitary pump stations provided some valuable data with which to estimate I&I from the 17 mi^2 area they serve. Estimates of daily discharge volumes were derived from pump run times and rated pumping capacities (Section 6.2).

The dry weather residential inflow was calculated for each pump station in terms of gpcd and compared to the industry standard, with values over 120 gpcd generally considered excessive (U.S. EPA, 1985). Based on this metric, 11 pump stations have excessive dry weather flow. (See Figure 29).

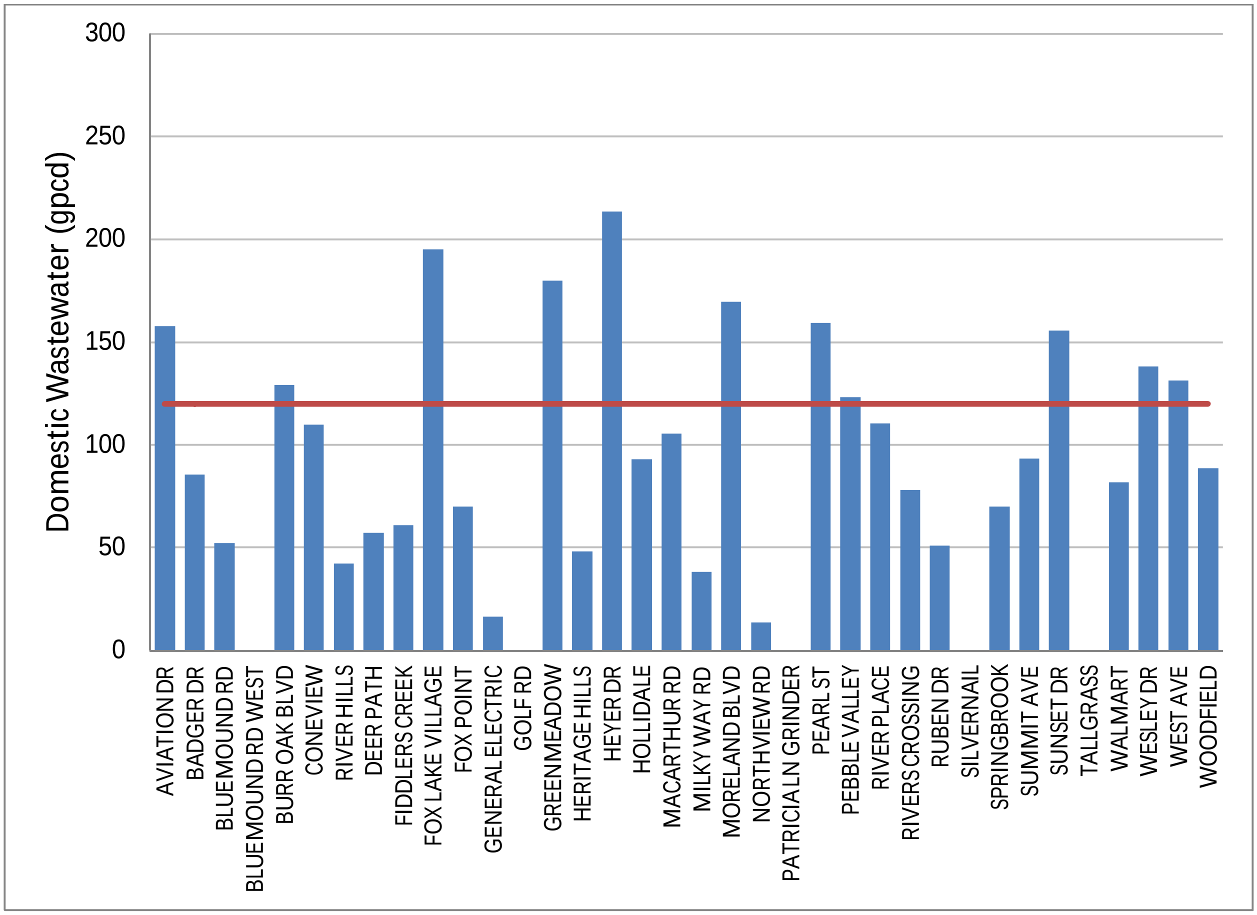


Figure 29 – 2008 Pump Station Dry Weather Flows

A significant limitation of the gpcd metric is that if the population served is spread over a large area, the per capita infiltration rates would be expected to be higher than for a more densely populated area due to the greater length of sewer required to serve the area with a lower population density. Normalizing infiltration by sewer volume is an alternate technique that remedies this limitation. Therefore dry weather pump station infiltration rates were also calculated as gallons per day per inch*diameter*mile (GPD/IDM) of sewer.

Infiltration rates were calculated by comparing station discharge rates calculated from run times to water consumption data for the area served with the difference being attributed to I&I. This approach assumes that 100% of consumption is discharged into the sewer system. In order to minimize error, water consumption data was limited to winter months, when irrigation is unlikely to occur.

Infiltration rates in excess of 3,000 GPD/IDM are generally considered excessive. (Indiana Department of Environmental Management, 1992) The results of this analysis are shown in Figure 30. Badger Drive, Greenmeadow, Heyer Drive, Silvernail, Sunset Drive, Wesley Drive and West Ave pump stations all exceed 3,000 GPD/IDM. Greenmeadow, Heyer Drive, Sunset Drive, Wesley Drive, and West Avenue are excessive using both methodologies.

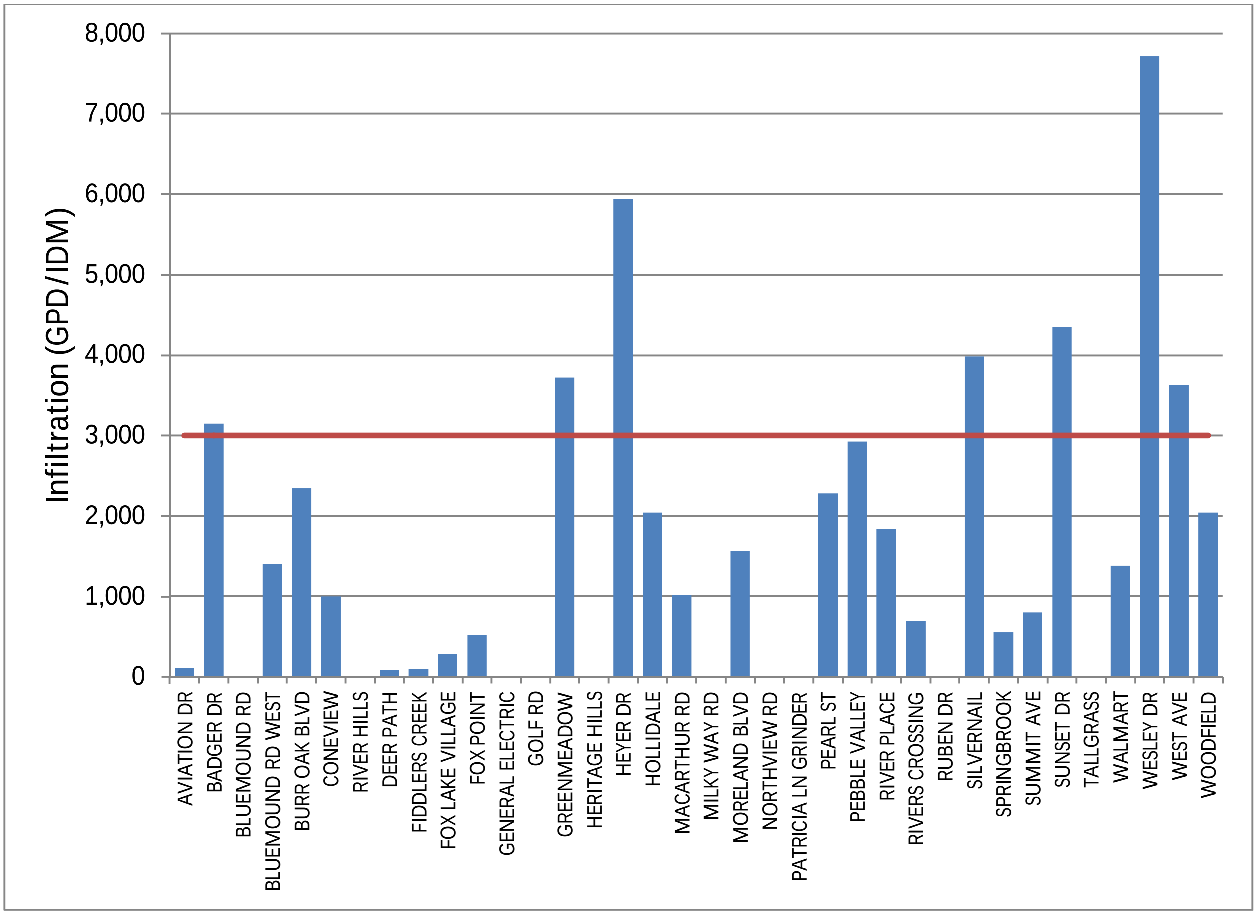


Figure 30 – Pump Station Infiltration Rates (GPD/IDM)

Since the data is normalized, the preceding figure does not indicate whether the infiltration rates, even if excessive, constitute a significant load on the City's collection system. Figure 34 in Section 4.5 indicates the magnitude of each of these pump stations' infiltration rates.

2008 pump station daily average peaking factors have been included as Appendix B.

4.4 2009 Flow Monitoring Program

To better understand the quantity and character of wet weather flows from the 8.4 mi^2 "unpumped" central area, the City implemented a flow monitoring program. This program also included installing 4 flow meters in the Pebble Valley service area in order to better understand where the wet weather flows are likely originating in this critical area. The Heyer Drive service area was re-evaluated to confirm the high infiltration rates that were calculated during the 2008 pump run time evaluation (Section 4.3).

4.4.1 FLOW METER LOCATIONS

ISCO 2150 area-velocity (AV) flow meters were installed at the 13 locations indicated in Figure 32. Detailed information about each site has been provided in Table 7. Site plans of each installation are included as Appendix C. In addition, ISCO pump station monitors were installed at the Heyer Drive and Coneview pump stations. These monitors utilize pump start/stop times and wet well geometry to calculate station inflows and outflows to a high degree of accuracy. Rain gauges were installed at the WWTP, Heyer Dr Lift Station and the Park Rec Lift Station (Figure 32). Appendix D summarizes the flow data collected at each site.

4.4.2 DRY & WET WEATHER PERIODS

Flow and rainfall data was collected from April 20, 2009 through July 24, 2009. Average daily rainfall totals from the 3 gauges are indicated below. Several significant rainfall events occurred, including one on April 25-26 (3.2”) and one on June 18-19 (4.7”). On a 24-hour basis, these storms had recurrence intervals of 1 year and 25 years respectively. Four of the largest measured storms are plotted against intensity-duration-frequency curves from Bulletin 71 (Midwestern Climate Center, 1992) in Figure 33.

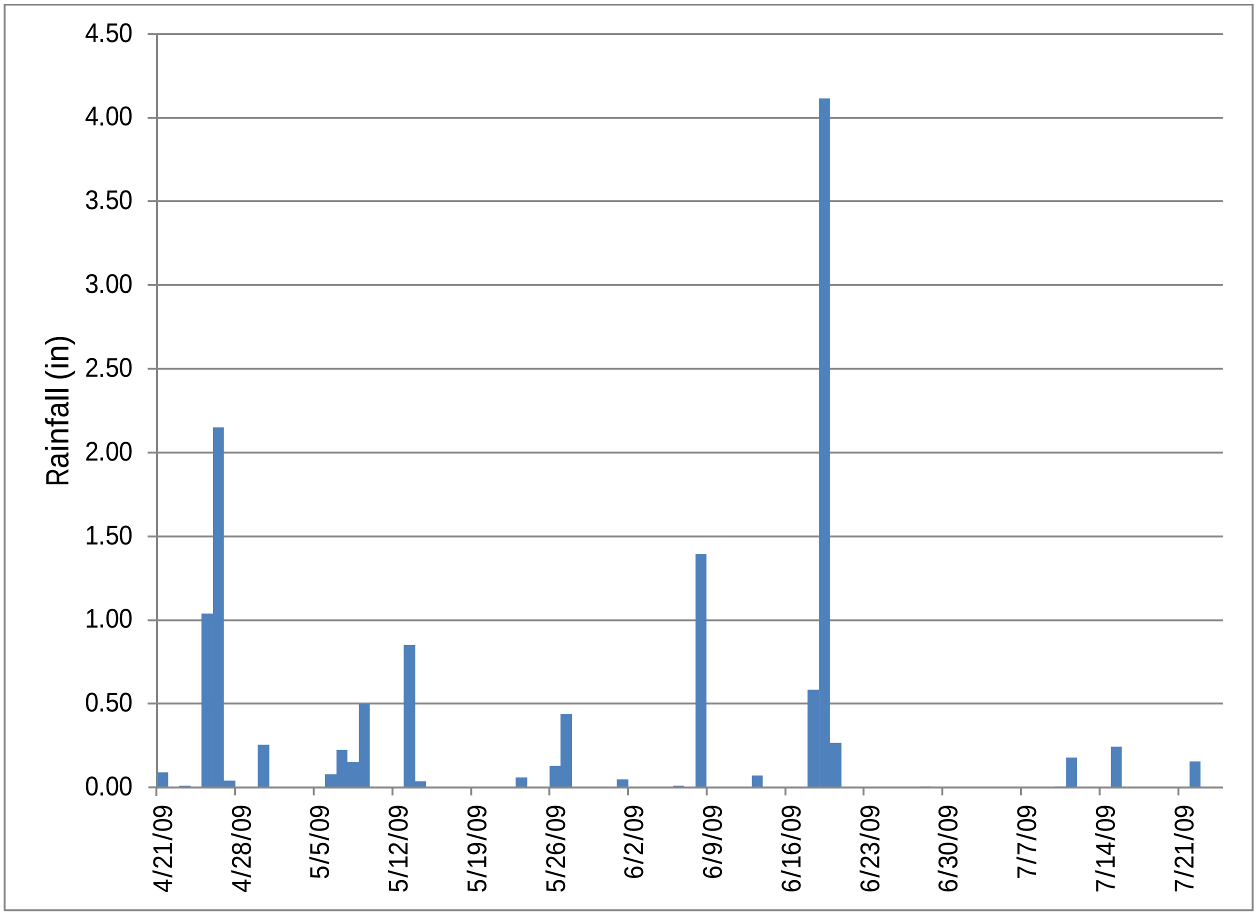


Figure 31 – Daily Rainfall Totals

Dry weather periods were evaluated to calculate base infiltration by averaging flow data for days in which little or no rain fell. These analyses excluded flow data from days following rain events if the metered flow remained elevated.

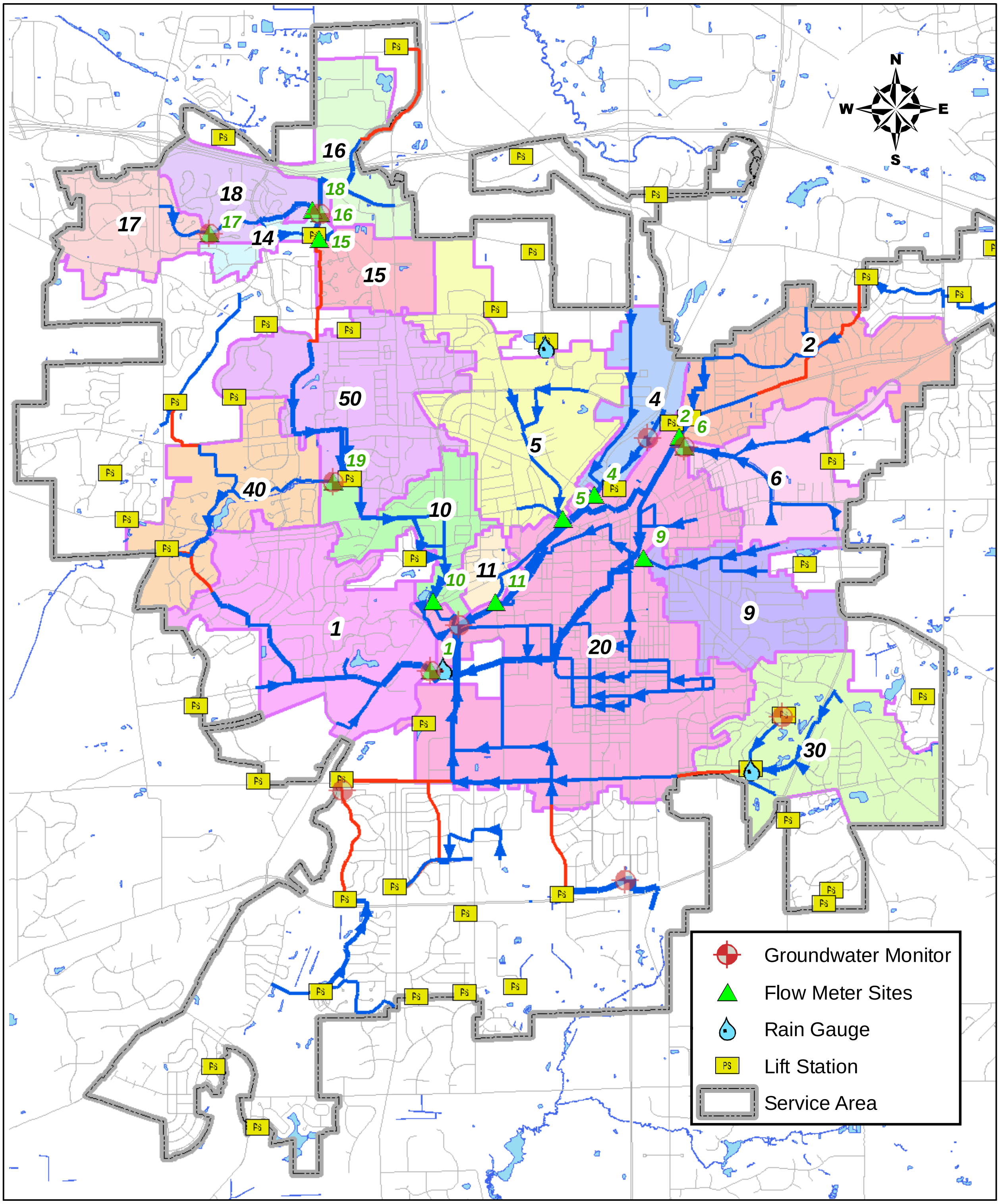


Figure 32 – Flow Monitoring Sites

Table 7 – Flow Monitoring Sites

Meter ID	Manhole ID	Pipe Size	Manhole Invert	Manning's n	Manhole Depth (ft)	Slope (%)	Down-stream Slope	Material	Qd (cfs)	Service Area (acres)	Upstream Area (acres)	Meter Error (%)*	Flow Accuracy	Comments
1	4141	30	784.41	0.013	18.7	0.147%	0.200%	RCP	15.74	887	2,033	10%	33%	Error likely higher since some Greenmeadow flow was transferred to Coneview.
2	3578	27	818.04	0.013	13.3	0.548%	0.400%	RCP	22.97	657	349	10%	15%	Upstream: Wal-Mart
4	1116	24	802.95	0.013		0.112%	0.582%	RCP	7.59	269	-	10%	10%	
5	1128	24	802.95	0.013	11.0	0.364%	0.413%	Clay	13.67	811	183	10%	12%	Upstream: Northview Rd
6	5632	21	822.55	0.013	7.6	0.369%	0.413%	RCP	9.65	441	114	10%	13%	Upstream: Ruben Dr
9	3374	15	814.15	0.013	10.0	1.394%	0.481%	Clay	7.64	472	106	10%	12%	Upstream: Pearl St
10	4917	18	825.51	0.011	11.2	0.588%	6.176%	PVC	9.54	454	70	10%	12%	US: Greenmeadow & Woodfield. Error likely lower since some Greenmeadow flow was transferred to Coneview.
11	1365	15	792.86	0.013	17.4	1.579%	0.239%	Clay	8.13					
19	2193	18	862.86	0.013	9.7	0.491%	0.139%	RCP	7.37	-	-	10%	10%	Greenmeadow to Coneview gravity bypass.
15	4404	15	873.03	0.013	10.0	0.524%	0.618%	RCP	4.68	243		10%	10%	Pebble Valley pump station service area
16	1596	21	864.93	0.013	13.0	0.673%	0.651%	RCP	13.02	369	183	10%	15%	
17	1385	12	930.13	0.011	11.0	2.793%	1.054%	PVC	7.05	334		10%	10%	
18	1600	21	864.78	0.013	16.6	0.061%	0.203%	RCP	3.91	260	334	10%	23%	

Sub-Total	**5,953**	**acres**
****Metered Area**	**3,991**	**acres**
Un-Metered Area	**2,031**	**acres**
Pumped Area	**7,000**	**acres**
Total Monitored Area	**10,991**	**acres**
Total Service Area	**13,022**	**acres**
% Monitored	**84%**	

* Meter Error = 10%
** Excludes monitoring sites 7,8,12, & 14. Not monitored due to large upstream area. Merged into WWTP sub-area.

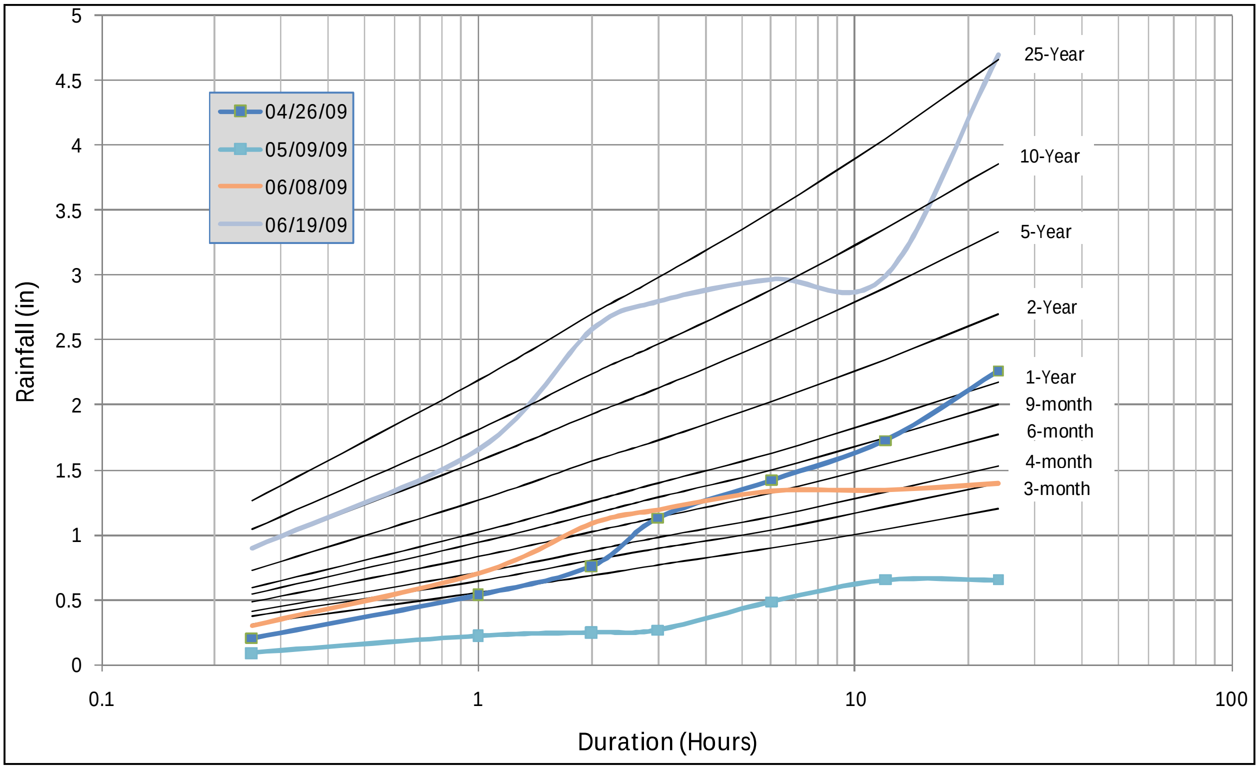


Figure 33 – Rainfall Distributions of Major Storms

4.5 BASE INFILTRATION ANALYSIS

Infiltration is typically estimated using spring flows when the seasonal groundwater table is high. Data following rainfall events was excluded until flow returned to pre-rainfall levels. Two industry standard metrics were used to quantify infiltration.

The first of these two metrics normalizes base flows by population. After discounting significant industrial/commercial flows, remaining flows in excess of 120 GPCD are generally considered excessive. The results of this analysis are included in Table 8, however this is not considered a reliable method of quantifying infiltration.

Alternatively, infiltration rates were also quantified using the GPD/IDM method described in Section 4.3. The results of this analysis are presented in Figure 34 and Figure 35. All infiltration flow computations have been included in Table 8.

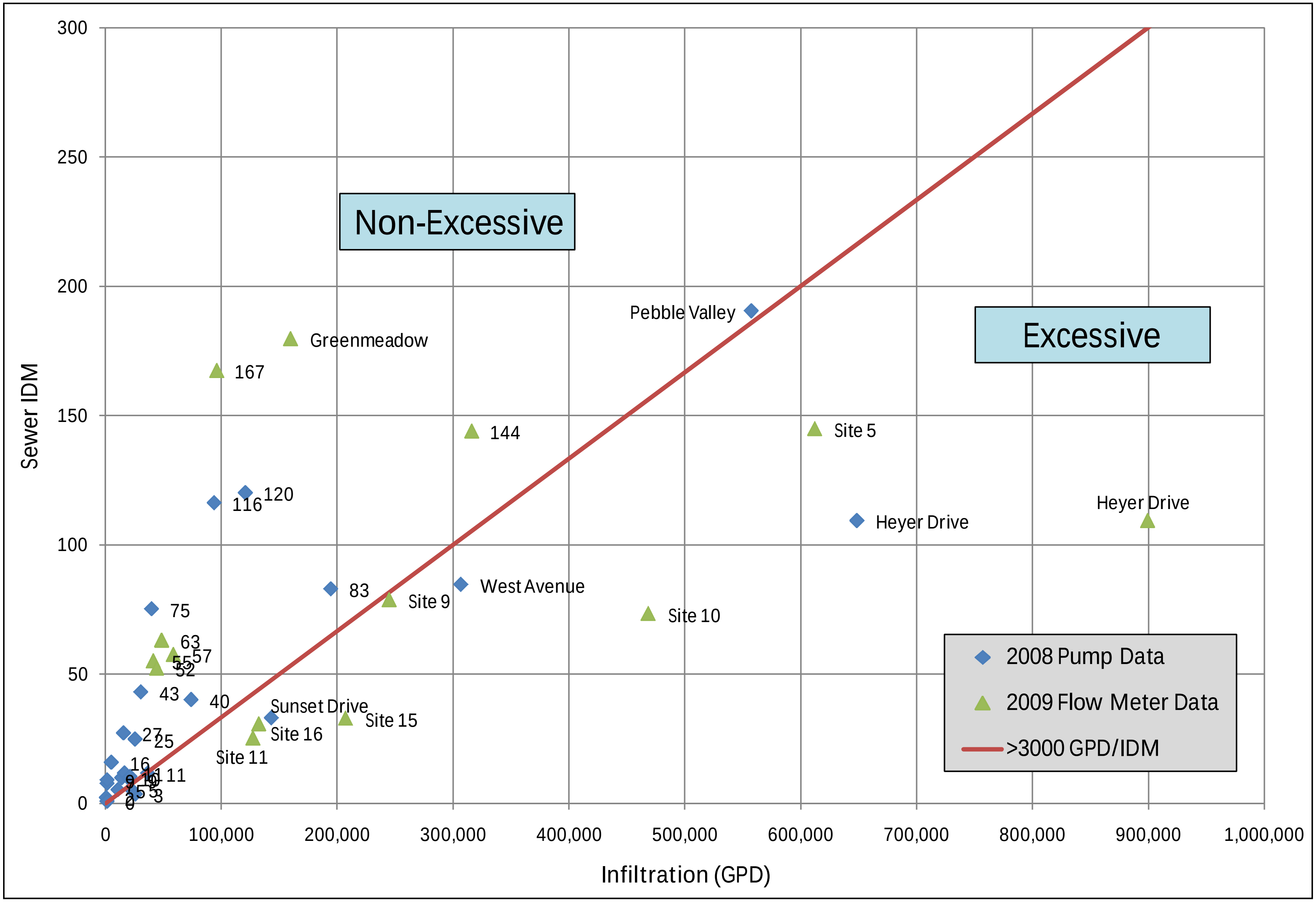


Figure 34 – 2009 Infiltration Rates (GPD/IDM)

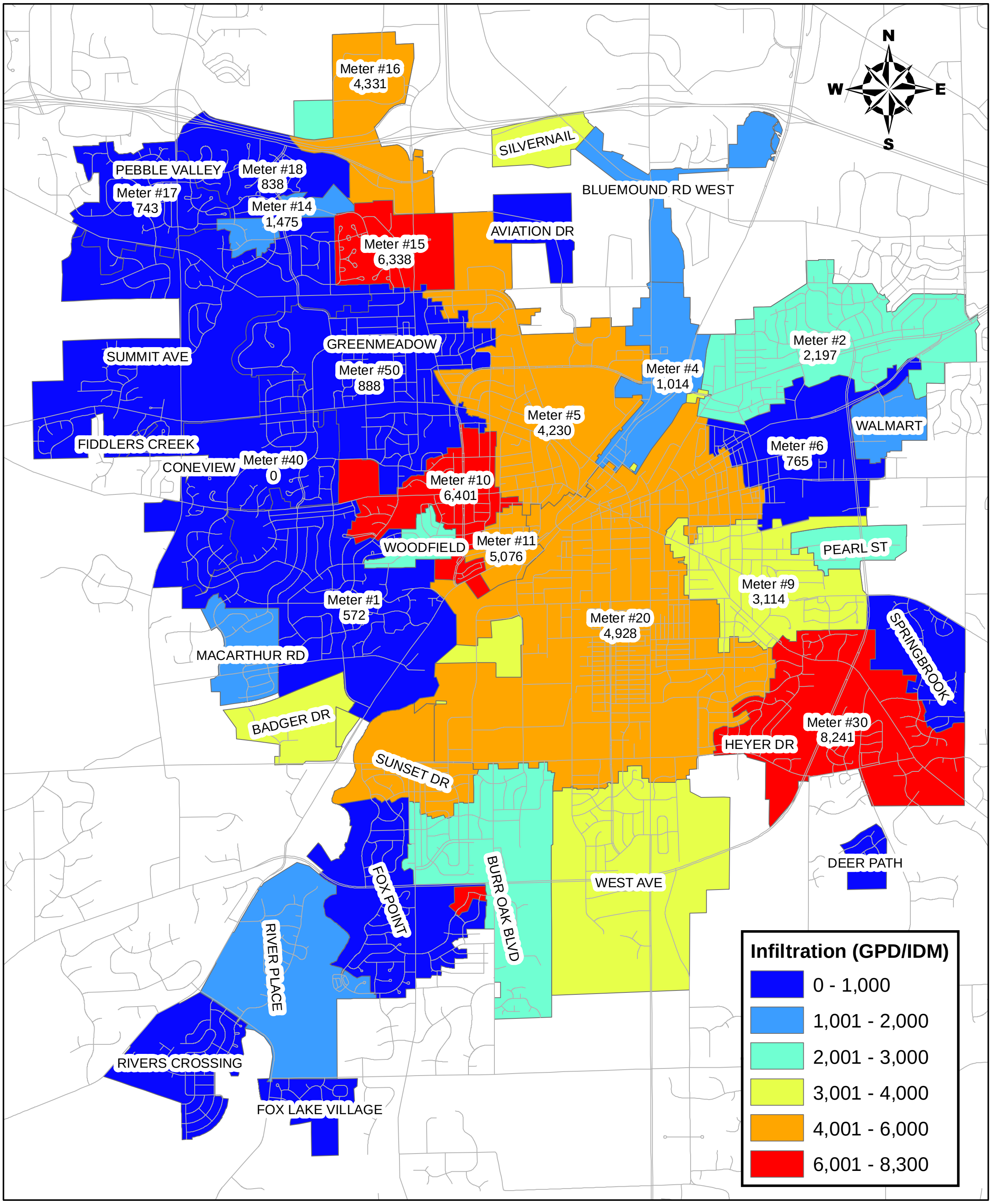


Figure 35 – 2009 Infiltration Map

Table 8 – 2009 Base Infiltration

Site ID	Tributary Areas	Total Area (acre)	Incremental Service Area (acre)	2000 Census Population	Flow-Based Population Estimate	Cumulative Dry Weather Flow (gpd)	Cumulative Sanitary Flow (gpd)	Cumulative PE's	Cumulative Residential Sanitary Flow (gpd)	Tributary Residential Sanitary Flow (gpd)	Tributary Dry Weather Flow (gpd)	Tributary Sanitary Flow (gpd)	Incremental Dry Weather Flow (gpd)	Incremental Residential Sanitary Flow (gpd)	Incremental Sanitary Flow (gpd)	Infiltration (gpd)	% Infiltration	Residential Dry Weather Flow (gpcd)	Sewer IDM	Infiltration (gpd/IDM)
1	Coneview, MacArthur Road		887	4,924	3,887	897,996		16,630			521,763		376,233	209,902	280,553	95,680	25%	79	167	572
2	Ruben Dr	1,006	657	3,478	3,425	780,523		14,454			195,617		584,905	184,935	269,073	315,833	54%	146	144	2,197
4	Aviation, Bluemound Road West	696	269	1,009	599	326,885		6,053			35,919		290,966	32,323	232,821	58,144	20%	151	57	1,014
5		811	811	4,608	4,109	909,247		16,838					909,247	221,884	297,338	611,908	67%	203	145	4,230
6	Walmart	555	441	832	1,015	351,970		6,518			46,523		305,447	54,793	257,376	48,070	16%	101	63	765
9	Pearl Street	565	459	2,699	2,569	531,222	297,193	9,837			30,600	10,370	500,622	138,701	286,823	244,399	49%	149	78	3,114
10	Woodfield	406	336	3,242	2,816	681,637		12,623			52,957	32,247	628,680	152,070	160,573	468,107	74%	220	73	6,401
11		93	93	330	450	207,380		3,840					207,380	24,310	80,506	126,874	61%	336	25	5,076
15		243	243	986	1,243	286,555		5,307					286,555	67,143	79,785	206,770	72%	220	33	6,338
16	General Electric	478	343	1,243	0	200,175	86,139	3,707			7,702	25,398	192,473	0	60,741	131,732	68%		30	4,331
17		334	334	854	815	86,805	46,065	1,607					86,805	44,016	46,065	40,740	47%	104	55	743
18	17, Golf Rd	668	287	1,395	2,268	254,095	169,764	4,705			86,805	46,065	167,290	122,490	123,699	43,592	26%	73	52	838
19						178,500		3,306					178,500							
14	15, 16, 18	965	93	0	600	801,377	288,353	14,840			740,825	255,903	60,553	32,374	32,451	28,102	46%	101	19	1,475
Greenmeadow	Pebble Valley	2,295	706	4,794	5,595	1,408,934	736,529	26,091	736,529	434,418	801,377	288,353	607,557	302,111	448,176	159,381	26%	82	179	888
Heyer Dr	Springbrook, Milky Way, Deer Path, Deer Trails	1,198	745	5,998	4,061	1,221,454	315,962	22,620	315,962	96,650	102,909	96,650	1,118,545	219,312	219,312	899,233	80%	275	109	8,241
Coneview	Summit*, Heritage Hills*, 19		556	4,331	2,155	484,704	257,913	8,976	257,913	141,537	412,377	141,537	72,327	116,376	116,376	-44,049	-61%	34	119	-370
20	1, 2, 4, 5, 6, 9, 10&11, Heyer Dr, West Ave, Burr Oak, Fox Point, Sunset Dr		1,999	14,342	13,201	11,704,000	6,410,000	118,704	3,943,000		7,454,064		4,249,936	712,840	1,410,816	2,839,121	67%	269	576	4,928
System Total					48,807	11,704,000	6,410,000	118,704	3,943,000				10,824,020	2,635,579	4,402,483	6,273,637	58%	186	2633	2,383

4.6 INFLOW ANALYSIS

Inflow, or peak wet weather flow rates, were calculated in terms of GPCD. Average wet weather flows greater than 275 GPCD are generally considered excessive (U.S. EPA, 1985).

However like infiltration, the per capita metric may be overly simplistic. Therefore, for each of the monitoring sites, peak wet weather flows were also quantified in terms of capture coefficient. The capture coefficient is defined as the fraction of the rainfall falling on a separated sewershed that enters the collection system as I&I. However this is more of a volumetric analysis rather than a peak flow analysis, and therefore does not completely characterize the nature of the wet weather response. The results of both analyses are presented in Figure 36 and Table 9.

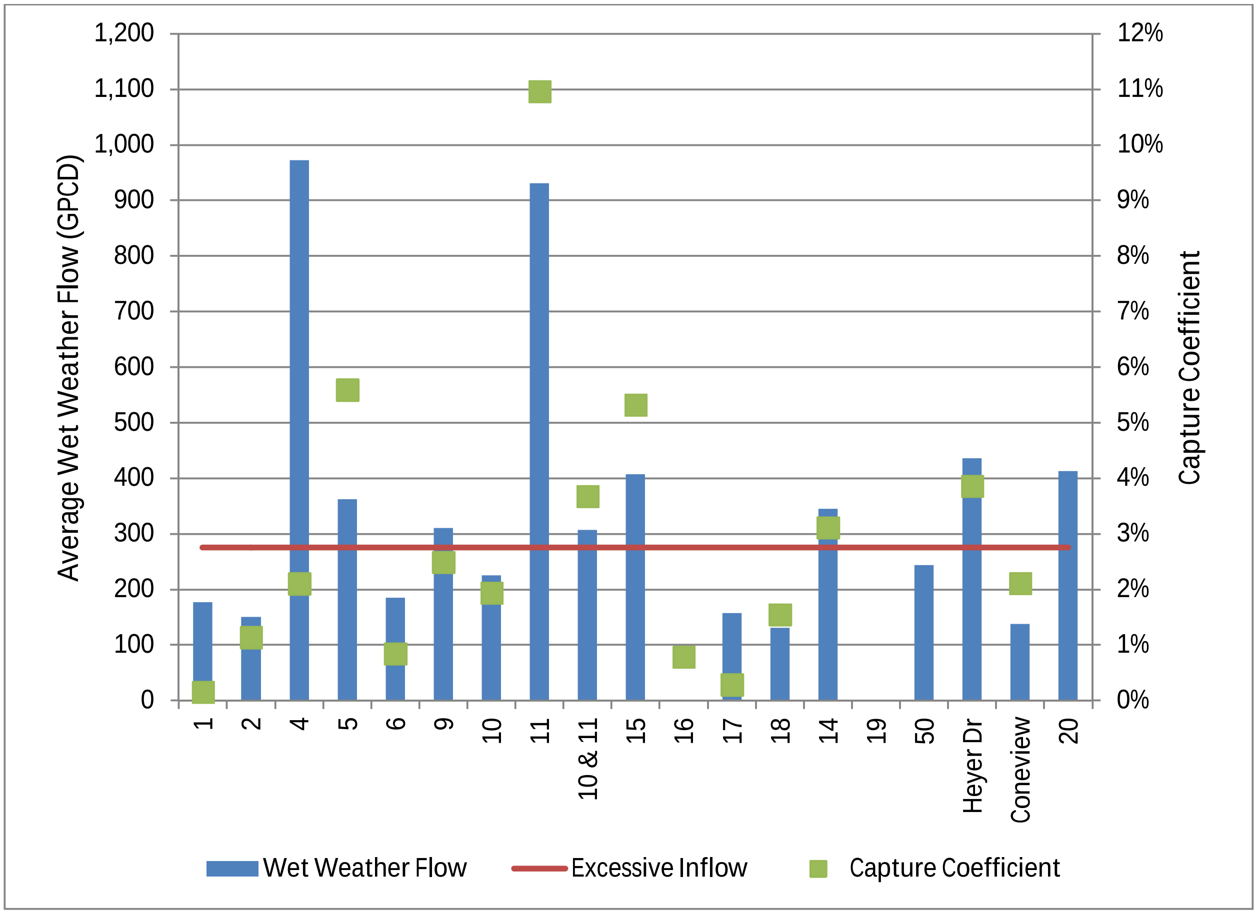


Figure 36 – Wet Weather Flow Analysis

Table 9 – Peak Wet Weather Flows

Site ID	Tributary Areas	Flow-Based Population Estimate	Incremental Service Area (acre)	Tributary Pump Station Population Estimate	Commercial Sanitary Flow (gpd)	Industial Sanitary Flow (gpd)	Other Public Sanitary Flow (gpd)	Tributary Pump Station Non-Residential Flow (gpd)	4/26/09 Wet Weather Flow (gpd)	5/09/09 Wet Weather Flow (gpd)	5/14/09 Wet Weather Flow (gpd)	6/08/09 Wet Weather Flow (gpd)	6/19/09 Wet Weather Flow (gpd)	Average Wet Weather Flow (gpd)	Average Wet Weather Flow Excl Com. and Ind. (gpd)	Peak Wet Weather Flow (gpcd)	
1	Coneview, MacArthur Road*	6,042	887	520	61,487	6,143	3,022	39,344			1,081,199	612,662	2,119,519	1,271,127	1,164,154	177	
2	Ruben Dr*	3,425	657	3,606	76,599	349	7,190	38,245	1,503,327	881,142	877,235	948,539	1,670,842	1,176,217	1,061,024	151	
4	Aviation, Bluemound Road West*	599	269	8	15,619	133,229	51,650	414	1,012,267	379,971	408,515		1,153,107	738,465	589,202	972	***
5		4,109	811		47,518	1,747	24,515		2,603,828	991,752	1,124,449	734,093	2,233,331	1,537,491	1,488,226	362	
6	Walmart*	1,015	441	526	50,924	151,659	0	4,108	601,083	356,714	402,631	377,411	720,882	491,744	285,054	185	
9	Pearl Street*	2,569	459	7	20,903	123,937	2,401	9,995	1,388,236	557,729	551,976	660,241	1,620,028	955,642	800,807	311	
10	Woodfield*	2,816	336	639	3,630	0	4,829	7	1,086,039	115,479	768,612	590,587	1,350,762	782,296	778,659	225	
11		450	93		53,413	2,756	0		1,296,059	154,850	86,741	54,799	783,491	475,188	419,019	931	**
10 & 11	Woodfield*	3,266	430	639	57,043	2,756	4,829	7	2,382,098	270,329	855,353	645,386	2,134,253	1,257,484	1,197,678	307	
15		1,243	243		10,020	0	2,622		902,628	291,061	424,667	245,442	717,834	516,326	506,306	407	
16	General Electric*	0	343	0	60,741	0	0	25,398	418,237	223,436	273,225	217,038	656,745	357,736	271,597		
17		815	334		0	0	2,049		155,350	114,858	104,162	81,319	184,138	127,965	127,965	157	
18	17, Golf Rd*	2,268	287	0	1,209	0	0	66,280	595,598	177,099	233,864	184,104	631,014	364,336	296,847	131	
14	15, 16, 18	600	965		72	0	0		207,155					207,155	207,082	345	
19		0							1,099,720		287,357	119,757	956,529	615,841	615,841		
50	Pebble Valley	5,595	706						2,348,728	733,053	986,512	764,769	1,991,917	1,364,996	1,364,996	244	
Heyer Dr	Springbrook*, Milky Way*, Deer Path*, Deer Trails*	4,061	745	1,790	11,449	0	1,812	203	3,777,750	1,594,355	2,109,502	1,634,113	3,710,723	2,565,289	2,553,636	436	
Coneview	Summit*, Heritage Hills*, 19	2,155	556	2,597	38,388	0	0	12,062	966,789		328,296	350,517	1,168,268	703,468	653,018	137	
20	1, 2, 4, 5, 6, 9, 10&11, Heyer Dr, West Ave, Burr Oak, Fox Point, Sunset Dr*	13,201	1,999	19,098	405,566	254,818	25,642	87,044	8,394,385	2,972,774	4,000,634	3,101,392	8,817,840	5,457,405	5,457,405	413	
System-Wide		54,228	10,561	29,430	914,579	677,393	130,561		30,739,277					20,966,169	19,838,517	366	

Estimated from model calibration mass balance.
Estimated from area's percentage of the total I/I for the 4/26 and 6/19 events.

*no wet weather flow for PS, so flow metric calculation area includes tributary pump station area
**flow split for 10 and 11 at several manholes. Most go to area 10 under average flow, but storm event would have pushed extra flow through meter 11.
*** PS are supplying additional flow, but only population of 8, 2000 Census shows 1,009 not 599 as calculated and area 4 is mostly industrial.

4.7 NEXT STEPS

The Heyer Drive PS service area consistently shows excessive base infiltration. Smoke testing is not effective for locating this sort of I&I. Costly sewer televising and/or dyed-water flooding are the most effective SSES methods to employ. In order to minimize the amount of testing, Donohue recommends that City personnel install the three of the four ISCO flow meters purchased during the 2009 flow monitoring program in the locations indicated in Figure 37. This will sub-divide this 745-acre area into 4 sewersheds (using the pump station as a meter). Once infiltration rates for each of these areas have been calculated, a focused SSES evaluation should be conducted with the intent of locating specific sewers contributing significant I&I and thereby likely requiring rehabilitation.

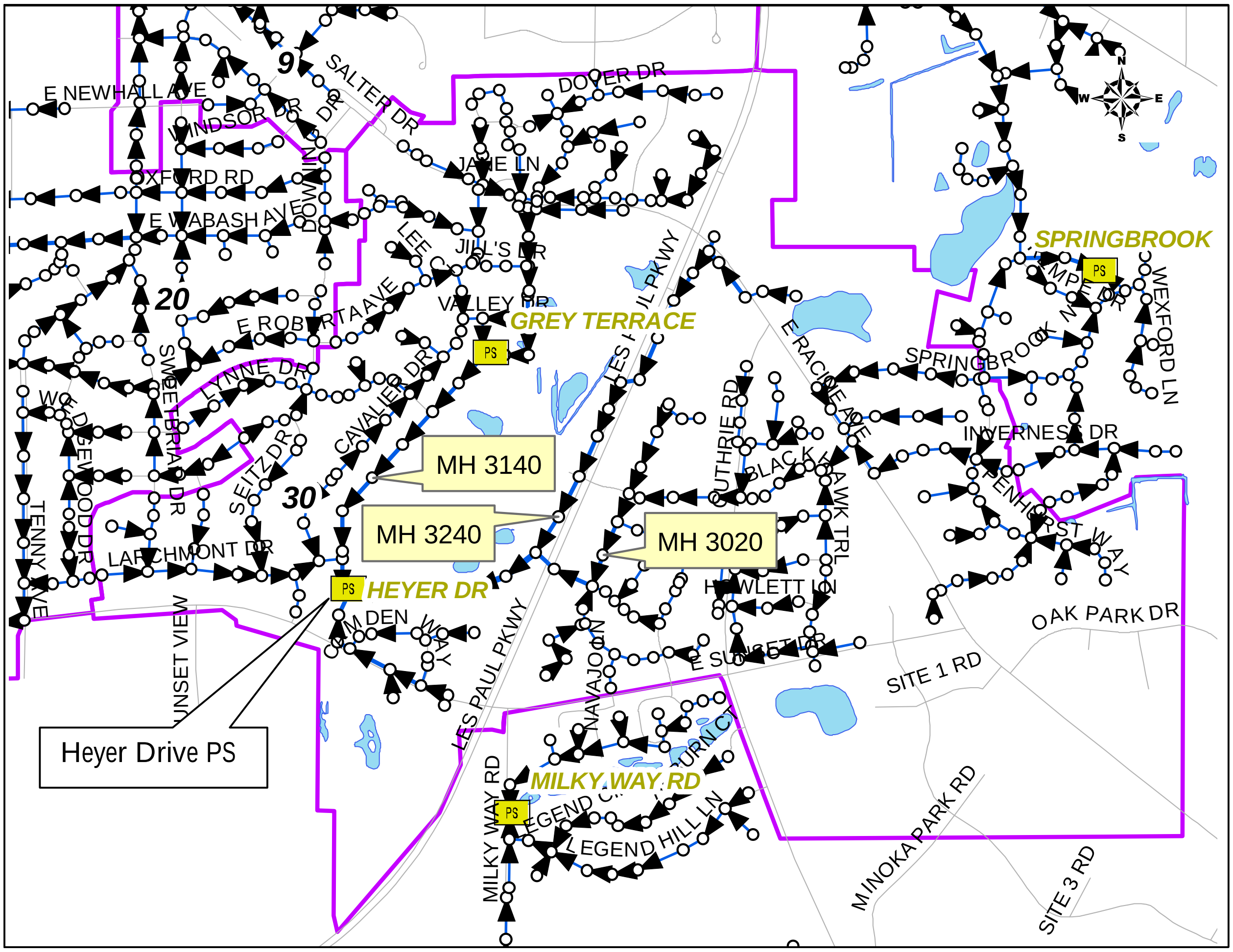


Figure 37 – Proposed Heyer Drive Flow Monitoring Sites

The branched configuration of the collection system, limited flow meter accuracy, and subsequent error propagation in flow mass balances make accurate determination of I&I rates from the central portion of the collection system impossible. However this is the oldest most low-lying part of the system. Furthermore, the 2009 flow monitoring study did estimate that I&I rates from this 2,000-acre area (#20 in Figure 32) are some of the highest in the system. (See Figure 35.) The limited flow monitoring data and SSES work that has been performed in this area has found it to contain some of the "leakiest" sewers in the system. Therefore Donohue recommends that an SSES program be employed to physically inspect all of the sewers in this part of the system, beginning with the oldest sewers. This is explained in more detail in Section 5.2.

CHAPTER V –SEWER SYSTEM EVALUATION SURVEY

While a comprehensive SSES has yet to be performed, Fall 2009 presented an opportunity to apply the inexpensive SSES technique known as smoke testing in those portions of the collection system where direct inflow appeared to be a significant portion of clear water flows.

5.1 SMOKE TESTING

Smoke testing is an inexpensive means of locating defects in the sanitary sewer system. During testing, a section of sanitary sewer is isolated and a harmless smoke is forced into the sewer system. The pressurized smoke will look for a means of escape. The majority exits the system via rooftop vent stacks, as should be expected. However it will also escape via illicit connections and structural defects. Smoke testing crews walk the length of the pipe being tested, locating, photographing, and logging any smoke escaping where it shouldn't.

While its low cost makes smoke testing an attractive SSES method, it does have significant limitations. The major limitation of this technique is that the smoke is unable to pass through water or saturated soils. Ideally testing is conducted when sewer catch basin traps are dry and groundwater is low to permit the maximum conveyance of smoke through defects.

Smoke testing is particularly effective at locating illicit sources of inflow such as directly connected down-spouts, inlets, and catch basins. However it can also locate structural defects as smoke emanating from the ground, often through cracks in the pavement near manholes. In some cases, it may locate structural defects in both the storm and sanitary sewer systems, as the smoke propagates through defects in these systems and the intervening soil matrix, ultimately escaping from catch basins and inlets.

5.1.1 TESTING AREAS

The areas and sewers that were tested are indicted in Figure 38. A total of 25.8 miles of sewer serving an area of 1,200 acres were tested. The Pebble Valley area was selected for testing because while the volume of I&I from this area is not particularly high, the suddenness of it makes it appear likely to be coming from direct connections that smoke testing is adept at locating. Furthermore, the operational challenges of conveying high flows from this area make it a high priority for I&I reduction.

5.1.2 LOCATED DEFECTS

Smoke testing was conducted by Visu-Sewer out of Pewaukee, WI. Their smoke testing logs have been included in their entirety in Appendix E. Photographs have been converted to GIS.

5.1.2.1 Pebble Valley

Figure 38 indicates the tested sewers and defects. Unfortunately, no significant sources of I&I were located.

5.1.2.2 Heyer Drive

Heyer drive experiences both excessive infiltration and a fast wet weather response indicative of direct inflow, and was therefore selected for smoke testing. Eleven defects were located, most of medium severity. Some of these are an indicator of sewers of suspect structural integrity.

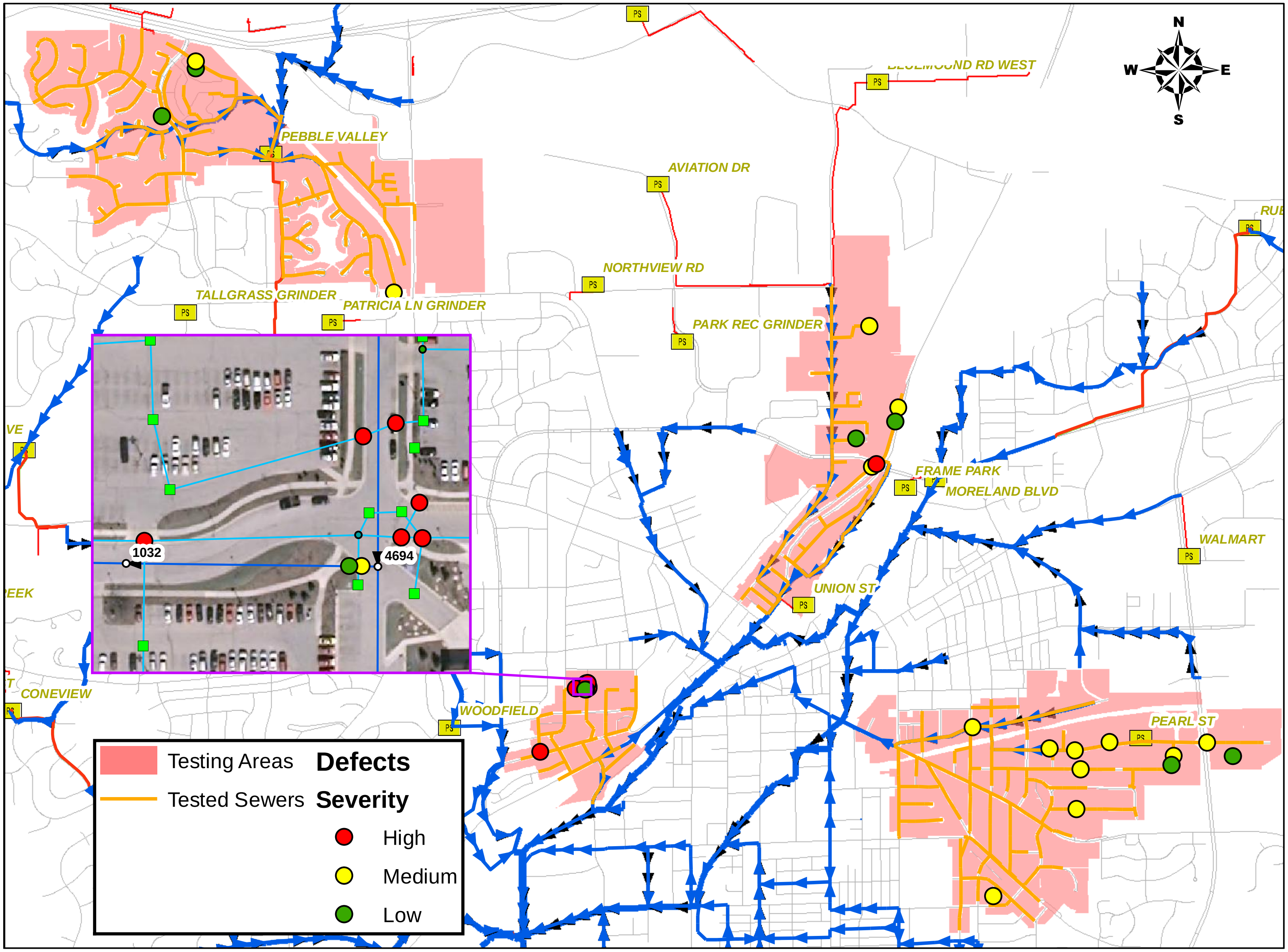


Figure 38 – Smoke Testing

5.1.2.3 Downtown

Quantifying I&I in the downtown area presented a unique challenge. This is because isolating the flows generated by these areas required subtracting out significant upstream flows. Limitations in meter precision can lead to error propagation, making the net flow generated by the downstream area highly questionable.

Nevertheless, wet weather flow data and the age of these sewers indicated a likelihood of excessive I&I being generated by the downtown area. However due to the uncertainty in the flow data, only a small 77-acre pilot area was selected for smoke testing. The most defects were found in this relatively small area.

Table 10 – Defects per Mile of Sewer Tested

Area	Length (miles)	# Defects	Defects / mile
Downtown	2.7	9	3.3
FM #4*	4.2	6	1.4
Heyer Dr	8.9	11	1.2
Pebble Valley	9.9	4	0.4

*Refers to the area monitored by flow meter #4 (Figure 35).

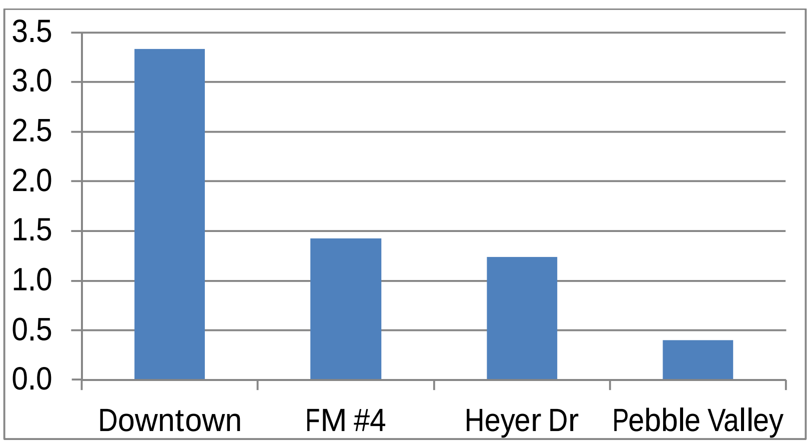


Figure 39 – Defects per Mile of Sewer Tested

5.2 Next Steps

A complete SSES program will be part of Phase II of this Sanitary Sewer Master Plan. Physical testing and inspection will identify and document problem areas and serve as a collection system baseline condition assessment.

Due to its low cost and success in locating defects in the downtown area, Donohue recommends that the City smoke test the remainder of this area in Summer 2010. The area to be tested is likely to be approximately 2,000 acres in size containing approximately 48 miles of sewer with 75% of those over 50 years old. The cost to test this area is approximately $75,000, however the City need not test it all in one year.

During the spring and summer of 2009 portable flow meters were installed at several locations to quantify infiltration and inflow and to calibrate the model. Donohue recommends Phase II flow monitoring at the three locations in the Heyer Dr area of the City, which experiences excessive I&I. The City does not maintain any permanent flow meters, though the pumping stations' SCADA systems can be used to continue to monitor flow.

In addition, the City has implemented a sewer televising program that will inspect 10-15% of the sewer system every year. At this rate, the entire 250 miles of sewer will be televised every 7 to 10 years. While additional flow monitoring in Spring 2010 will better isolate specific areas in the Heyer Drive service area contributing excessive I&I, Donohue recommends that Waukesha conduct spring sewer televising and perhaps dyed-water-flooding of those sewers where excessive infiltration is most likely originating. This would include older sewers, sewers more likely submerged by groundwater, and/or sewers crossing or adjacent to surface waters (creeks, ditches, etc.)

CHAPTER VI – COLLECTION SYSTEM MODELING

6.1 SOFTWARE SELECTION

After considering collection system modeling packages currently available, Donohue recommended utilizing MikeUrban by the Danish Hydraulic Institute (DHI) for this project. It has all the functionality the City requires, integrates well with GIS packages, and has a free graphical post processor, enabling Donohue to submit electronic simulation result files for review by City personnel. Donohue is certain that MikeUrban meets the City's short and long-term needs.

At the project Kick-Off Meeting, City personnel made it clear the City did not wish to take ownership of the software at this time, rather that the model be developed in a commercial package in a manner that would make it readily transferrable among modelers. Donohue considered XP-SWMM, SWMM 5.0, and MikeUrban for this project. All 3 packages use similar hydraulic computational engines and are all capable of predicting the response of the City's collection system to dry and wet weather flows. They differ primarily in their hydrologic model methods and their user interfaces. MikeUrban is in fact not a model, but rather a "model manager"; in other words, it is a GIS-centric interface for model development and post-processing, while capable of using either the MOUSE or SWMM 5.0 computational engines.

One of the advantages of MikeUrban is that it can utilize either the EPA-SWMM 5.0 or MOUSE hydrologic / hydraulic computational engines. It has the ability to simulate virtually any hydraulic phenomena including open channel flow, surcharge, pump station hydraulics, pressurized flow, flow splits, etc. Built upon ESRI's ArcObjects, it integrates with ESRI's ArcGIS platform. Finally, it's powerful and user-friendly graphical post-processor, MikeView, is freeware, which provides the ability to share model results with an unlimited number of people. Some of MikeUrban's features are:

- Steady-state or fully dynamic,
- Full pump and force main hydraulics,
- Open channel flow,
- Sewer surcharge and backwater effects,
- GIS integrated,
- Free graphical post-processor,
- Hydrology and wet weather impacts,
- Rainfall dependent inflow and infiltration (RDII) module,
- Long-term simulations,
- Scenario manager,
- Intuitive user-friendly interface,
- Daily, weekly, and monthly diurnal flow variations,
- Water distribution modeling (EPA-NET), and
- Load allocation.

Table 11 – Collection System Modeling Software Comparison

	XP-SWMM	SWMM 5.0	MikeUrban	
			MOUSE	SWMM
Time variable hydrology	●	●	●	●
Steady-state hydraulics	●	●	●	●
Dynamic hydraulics	●	●	●	●
Pressurized / surcharged flow	●	●	●	●
Ability to Copy Figures To Windows Applications		●	●	●
Automatic Reloading of Complex Result Windows			●	●
GIS Compatible (Links to ODBC databases)	Fair		Good	Good
Integration With ArcGIS 9.2			●	●
Linkage to External Databases	●	●	●	●
Complex structure (weirs, pump stations, etc)	●	●	●	●
Model Error Fixing Tools	●		●	●
Continuous I/I Analysis			●	
Real Time Control	Good	Fair	Good	Fair
Long Term Simulations, Extreme Statistics			●	
Water Quality	●	●	●	●
Sophisticated Dry Weather Flow Generation			●	
Dirunal Flow	●		●	
Groundwater	●		●	
Snowfall Accumulation / Melt	●	●	●	●
Infiltration / Rain Induced Infiltration	Poor	Poor	Good	Poor
Inlet Control	●		●	
Ability to View Results As Model Runs	●		●	
Unit Hydrograph	●	●	●	●
Intuitiveness / Ease of Use	High	Fair	High	High
Model Stability	High	Fair	High	Fair
GUI	Fair	Fair	Good	Good
Graphical Post Processor	Poor	Poor	Excellent	Excellent
Free Graphical Post Processor		●	●	●
Public Domain (Free)		●		

6.2 PUMP STATION HYDRAULIC EVALUATION

The model simulates pump stations using either their rated capacities, or using full pump hydraulics. The latter method calculates pump discharge as a function of head loss across the pump, simulating static and dynamic losses. While more precise, it is far more computationally intensive, therefore it is preferable to simulate pumps at their rated capacities if possible. For most of these pumps, only the pump performance curves were available, however a system curve is also required to locate the pump's operating point.

Discharge rates for each pump station were determined by performing hydraulic analyses of each of the stations. Data for each station and force main was compiled and a system head curve derived. The system head curves were plotted with the pump curves to identify the operating points for each number of pumps operating at each pump station (Appendix F). The results of this analysis are summarized in Table 12.

6.3 HYDRAULIC MODEL DEVELOPMENT

The 877-node model is a skeletal representation of the collection system, generally consisting of pipes 10 inches and larger, although smaller pipes are included if they are essential to properly routing of flows. The GIS was the primary source of information making the preparation of the model network fairly straightforward. However pump stations did present a challenge. Less significant stations were not included, the full pump hydraulics of the Greenmeadow pump station were simulated, and the following stations were simulated using their rated capacities: Burr Oak, Coneview, Fox Point, General Electric, Heyer Dr, Pebble Valley, River Place, Ruben Dr, Summit Ave, Sunset Dr, West Ave.

6.3.1 MODEL NETWORK

Including every manhole along each modeled pipe would increase computational times while not improving accuracy. Therefore the model was “skeletonized” by merging multiple pipe segments of relatively uniform size and slope into a single pipe segment. The completed model network is indicted in Figure 40.

Those pump stations that were included in the model were replicated using one of two methods. The easier of these two, used for the majority of modeled pump stations, presumes that each pump operates at its rated capacity. This method presumes that static and dynamic head losses are relatively uniform throughout the each pump’s operating range and that it does not deviate significantly from its design operating point. Each pump’s rated capacity was taken from Table 12.

For the Greenmeadow pump station, it appeared force main hydraulics might significantly influence pump performance, therefore these were simulated as head-discharge pumps. For these the model calculates pump discharge as a function of head loss across the pump, utilizing each pump’s performance curve while calculating static and dynamic head losses. Minor losses were assumed to be negligible. Absent pump performance testing results, each pump was presumed to be operating “like new”.

The WWTP was also included in the model. It was represented simply as a pump station, to replicate the potential hydraulic impact the primary pumps might have on the collection system.

Table 12 – Pump Station Hydraulic Evaluation

Modeled Pump Stations

Pump Station	GIS Ground El	Ground El	Datum**	Wet Well Floor El	Influent Pipe Invert	Wet well area (sqft)	Equival MH Dia (ft)	Pump Discharge El	FM Discharge El	Force Main L (ft)	FM Dia (in)	Force Main Material	Pump Quantity	Pump 1 On El	Pump 1 Off El	Pump 2 On El	Pump 2 Off El	Pump 3 On El	Pump 3 Off El	Pump Design Flow (gpm)	Pump Design Head (ft)	Pump curve on file	Q1 (gpm)	Q2 (gpm)	Q3 (gpm)	Comments
Burr Oak	801.8	21.00	780.80	-7.93	4.10	134	13.06	-7.10	804.188	5,543	12	DCI/CI	3	0.39	-4.30	1.05	-4.30	1.73	-4.30			no	975	1,400	1,575	
Coneview	846.9	65.47	781.43	43.00	53.90	134	13.06	43.75	863.188	2,562	16	DCI	3	51.00	47.67	51.67	47.67	52.33	47.67	1122	44.9	yes	1,122	1,975	2,533	Q1, Q2, Q3 in data from client
Fox Point	796.5	15.00	781.50	-15.00	-5.60	134	13.06	-14.50	804.188	8,184	16	PVC	3	-8.83	-10.50	-7.25	-10.50	-7.00	-10.50	1090	66	yes	1,425	2,140	2,530	
River Place	798.8	17.00	781.80	-15.95	-8.00	103.2	11.46	-15.20	795.558	405	10	PVC	2	-11.45	-12.95	-11.20	-12.95			580	23.5	yes	475	900	n/a	
Ruben Dr	880.6	19.00	861.60	0.00		104	11.51	2.50	881.298	6,511	12	DCI	2	5.33	3.12	5.70	3.12			1015	45	yes	1,025	1,415	n/a	
Sunset Dr	793.4	793.80	n/a	774.50		96	11.06	775.00	804.188	3,831	8	CI	2	780.67	779.00	782.25	779.00			500	60	yes	525	600	n/a	
General Electric	871.4	91.00	780.40	68.20	78.45	62.5	8.92	68.80	880.298	5,034	8	DCI	2	74.41	71.20	74.91	71.20			430	50	no	375	460	n/a	
Greenmeadow	875.9	95.80	780.10	77.33	82.20	240	17.48	80.00	790.048	1,500	16	DCI	3 (VFDs)	81.67	80.17	82.17	79.83	82.50		2000	64	yes	2,900	4,300	5,000	Q1 at startup 1,800. Q1 700 rpm 2,950
Heyer Dr	853		n/a	823.00		134	13.06		905.758	2,656	14	DCI	3	829.17	827.50	830.75	827.50	831.00		1500	103	yes	1,825	2,650	3,000	
Pebble Valley*	883	883.00	n/a	853.75	863.15	134	13.06		987.258	4,154	16	DCI	3	859.9	858.3	861.5	858.3	861.8	858.3	1400	145	yes	1,400	2,300	2,750	
Summit	849.8	69.00	780.80	35.80	45.00	120	12.36	36.30	882.428	2,324	12	DCI	2	44.00	41.00	44.50	41.00			1291	79.8	yes	1,325	2,125	n/a	
West Ave	813.3	32.83	780.47	16.00	21.70	115.3	12.12	18.00	841.798	3,301	10	CI	3	19.2	17.4	20.8	17.4	21.0	17.4	810	64	yes	775	1,100	1,250	

* Pebble Valley wet well El based on Fox Point Influent pipe invert 9.4 ft above wet well floor (Peb Valley Infl pipe invert 863.148) and same wet well area and same operating points relative to wet well floor.

** GIS Datum is 780.558

Non-modeled Pump Stations

Pump Station	Force Main Discharge Elev	Force Main L (ft)	Force Main Dia (in)	Force Main Materail	Pump Quantity	LWL El	HWL El	Q1 (gpm)	Q2 (gpm)	Q3 (gpm)	Comments
Aviation Dr	923.78	4,980	4	PVC	2	881.77	882.50	70	80	n/a	
Badger Dr	817.11	4,690	10	DI/HDPE	2	780.31	786.06	750	925	n/a	
Bluemound	896.06	516	4	DI	2	867.70	870.89	185	205	n/a	
W. Bluemound	923.81	4,732	10	PVC	2	850.39	852.73	470	640	n/a	
Corporate Dr	799.91	5,671	10	PVC	2	782.85	787.32	850	1025	n/a	
Dana(River Hills)	901.89	1,546	4	PVC	2	838.80	841.13	86	102	n/a	
Deer Path	944	1,093	4	PVC	2	901.69	903.02	86	97	n/a	
Deer Trails		642	4	PVC							
Fiddlers Creek	838	1,025	4	PVC	2	825.97	826.97	100	111	n/a	
Fox Lake Village	788.03	3,960	6	HDPE	2	764.56	767.56	240	275	n/a	
Golf Road		1,100	6	PVC							
Heritage Hills (Madison PS)	855.13	1,816	8	PVC	3	804.74	810.24	395	525	n/a	
Hollidale	883.53	68	4	CI	2	870.84	874.90	440	680	n/a	
MacArthur Rd	834.82	2,279	12	DCI	2	787.56	793.31	950	1450	n/a	
Milky Way	847.68	1,277	8	PVC/DCI	2	826.47	827.68	460	620	n/a	
Moreland Blvd	n/a	n/a	n/a	n/a	n/a	n/a	n/a	30	n/a	n/a	grinder pump(6" concrete gravity line)
Northview	919.07	713	6	CI	2	891.56	892.56	350	470	n/a	
Pearl St	857.52	1,436	8	PVC	2	834.06	837.06	340	455	n/a	
Rivers Crossing	784.7	3,866	8	PVC	2	777.64	779.97	445	520	n/a	
Silvernail	906	3,054	6	PVC	2	852.33	853.99	165	200	n/a	
Springbrook	866.7	4,056	10	DI	2	843.56	847.81	695	890	n/a	
Wal-mart	877.7	1,201	10	DI	2	839.56	843.81	700	1075	n/a	
Wesley Dr	903.83	1,682	4	PVC	2	842.74	845.07	88	103	n/a	
Woodfield	840.558	695	4	DCI	2	831.39	832.39	122	140	n/a	

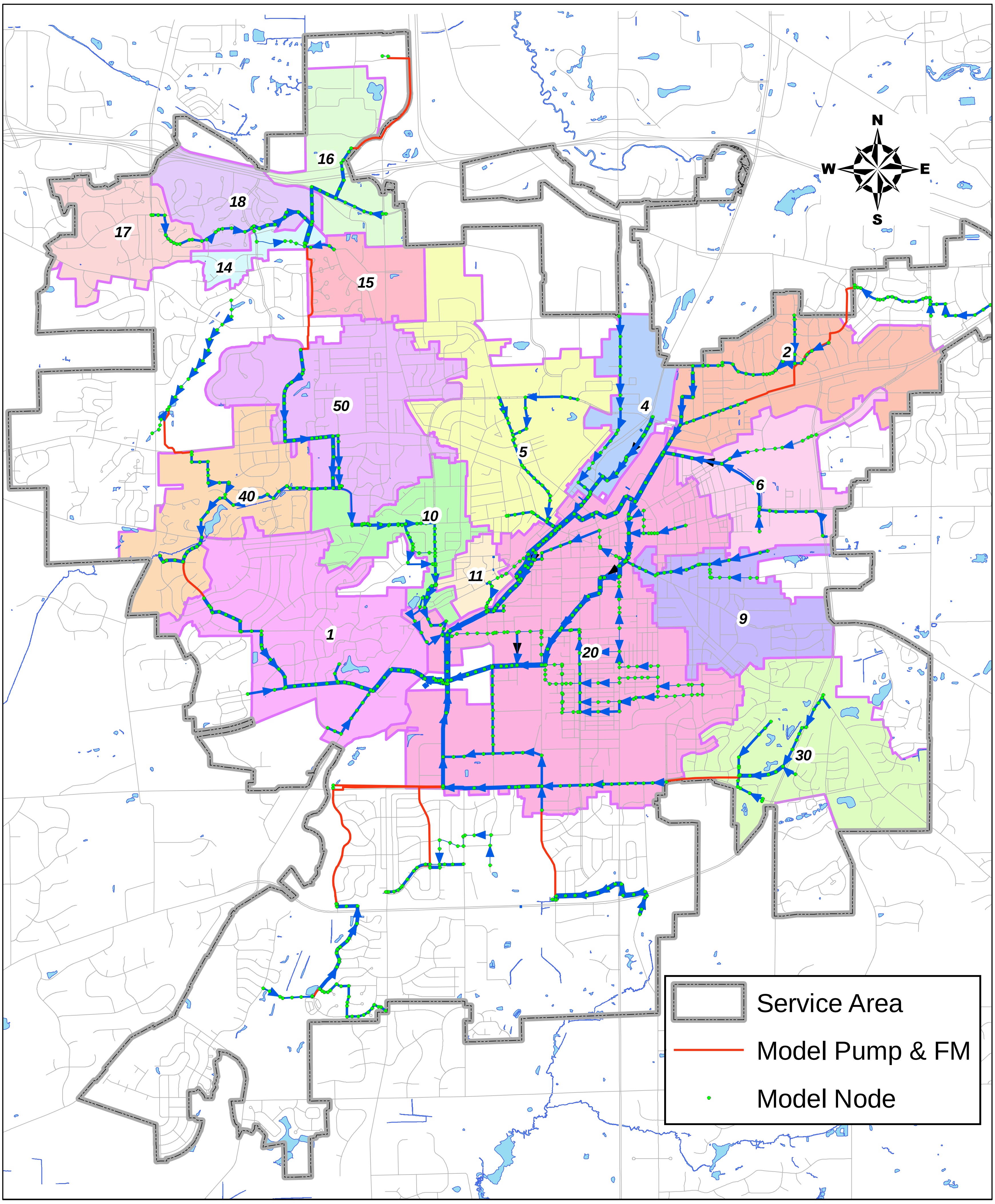


Figure 40 – Hydraulic Model Network

6.3.2 DRY WEATHER FLOWS

This being a sanitary sewer system, it is vital to simulate wastewater flows to a high degree of precision. Therefore water billing records were used as the primary source of dry weather flows into the model. This method presumes that 100% of water consumption is discharged into the collection system as wastewater. Winter billing records were selected so as not to include irrigation water in the evaluation. Water billing records were "geocoded" (located by address) and assigned to the nearest model node (Figure 41).

Flows were also categorized as residential, commercial, and industrial according to how it is reported in the water billing records. Diurnal flow distributions appropriate for each category of flow were assigned to the particular wastewater flow component. Residential flows were distributed using a typical residential 24-hour diurnal curve, industrial flows were presumed to be relatively constant over a 24-hour basis, and commercial flows were presumed to be discharged during 9 working hours per day, and to cease the remainder of the day.

Infiltration rates were constants derived during the infiltration analysis (Section 4.5), and distributed among all the point loads according to each load point's wastewater load. The result is a model that very accurately predicts dry weather flows for each pipe included in the hydraulic model.

6.3.3 WET WEATHER FLOWS

The City was fortunate to have flow meters installed and collecting data during two significant rainfall events. (See Section 4.4.2.)

The April storm was a one-year event that occurred in the spring during a period of high groundwater and soil moisture content. The June storm was a 25-year event, and while it occurred following a relatively dry period hence the soil had high absorptive capacity, it came in two waves, with the second wave falling on soil that had been saturated during the first wave. Both storms resulted in peak flows to the WWTP in excess of 48 MGD, the only other time that has occurred in the past 10 years was June 8-10, 2008, a 100-year storm.

The wet weather flows measured by the flow meters were input directly into the model, often using a series of mass balances to "back out" flows from upstream meters from downstream meters to quantify the intervening flow. Modelers sometimes had to adjust model inflow to account for the attenuation that may have been caused by capacity limitations. Wet weather flows were distributed throughout the flow meter sewersheds in proportion with the distribution of dry weather loadings generated from water billing records.

6.3.4 MODEL CALIBRATION

The model initially presumed the City collection system consists of clean pipes in good condition. Minor adjustments to pipe hydraulic parameters, primarily manning's roughness coefficients, were sometimes required so as to obtain a better match between measured and simulated water levels.

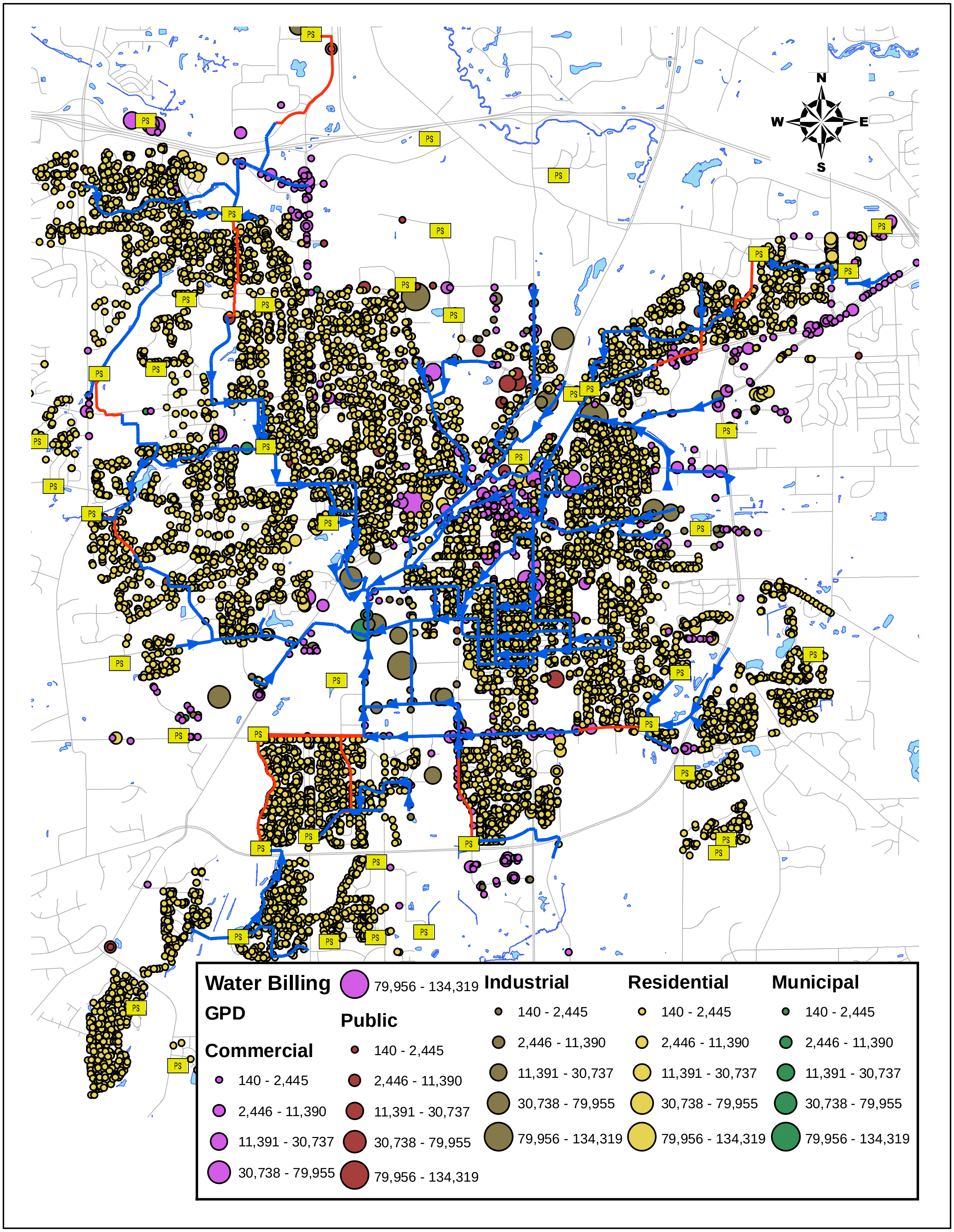


Figure 41 – Water Billing Records

Hydraulic parameters were never adjusted beyond a reasonable range so as to force a better fit between the measured and simulated hydraulic response. In those instances where a good calibration could not be achieved, there may be an idiosyncrasy in the collection system that is preventing a good calibration.

The following three scatter plots indicate a generally good agreement between measured and simulated peak flows, volumes, and water levels. Hydrographs and rainfall hyetographs of the flow meter data used for the two calibration events have been included as Appendix G. Time series graphs comparing measured and simulated flows and depths have been included as Appendix H.

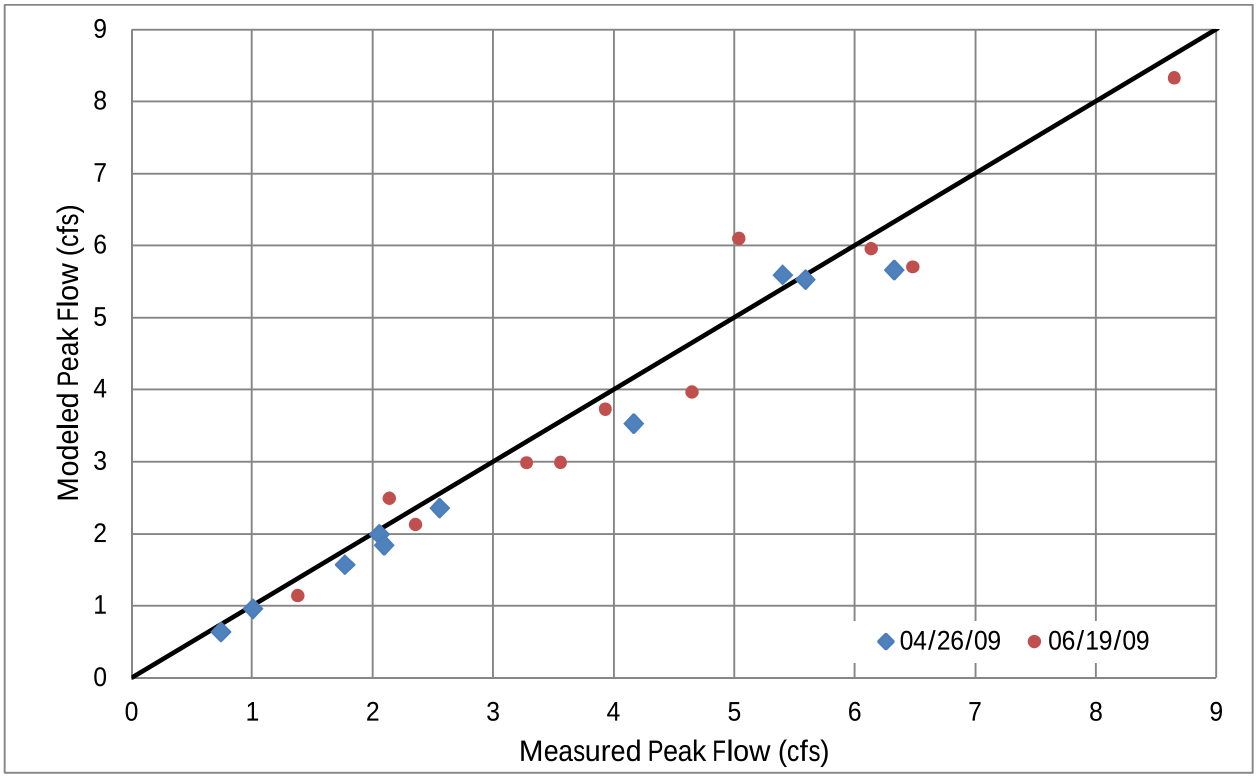


Figure 42 – Model Peak Flow Calibration

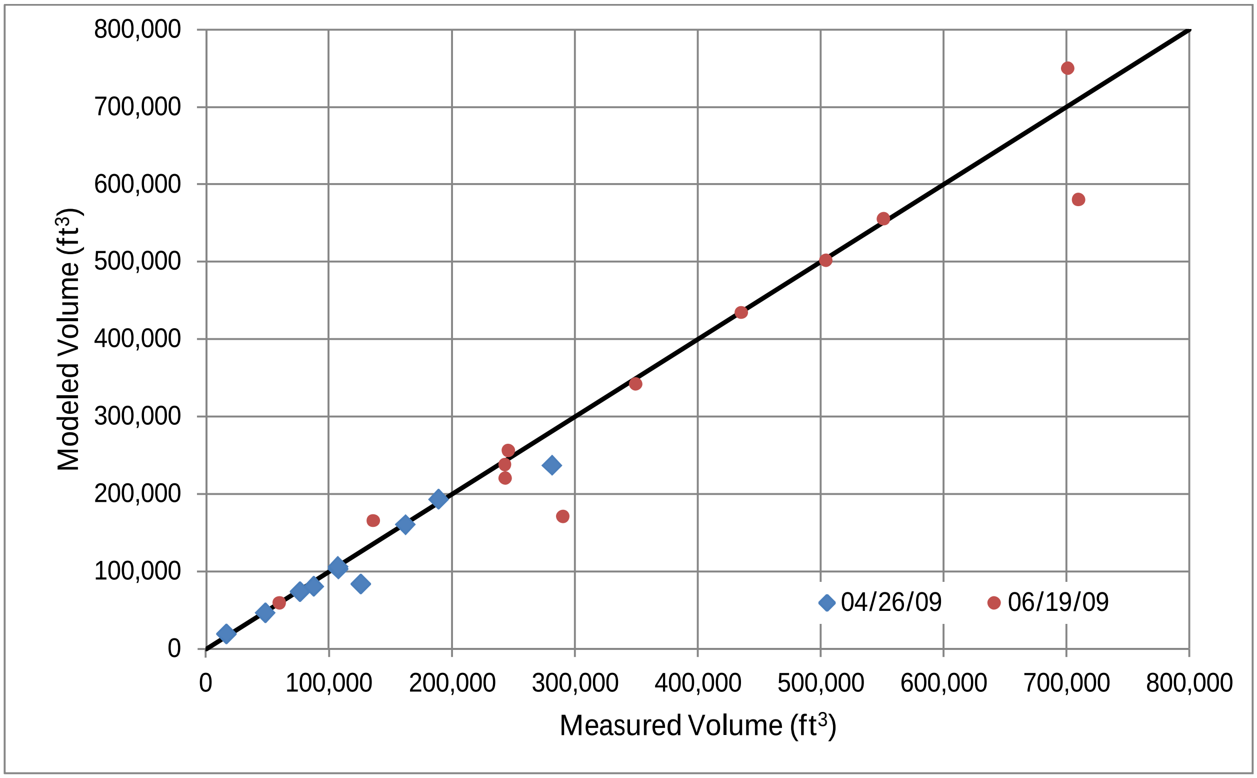


Figure 43 – Model Volumetric Calibration

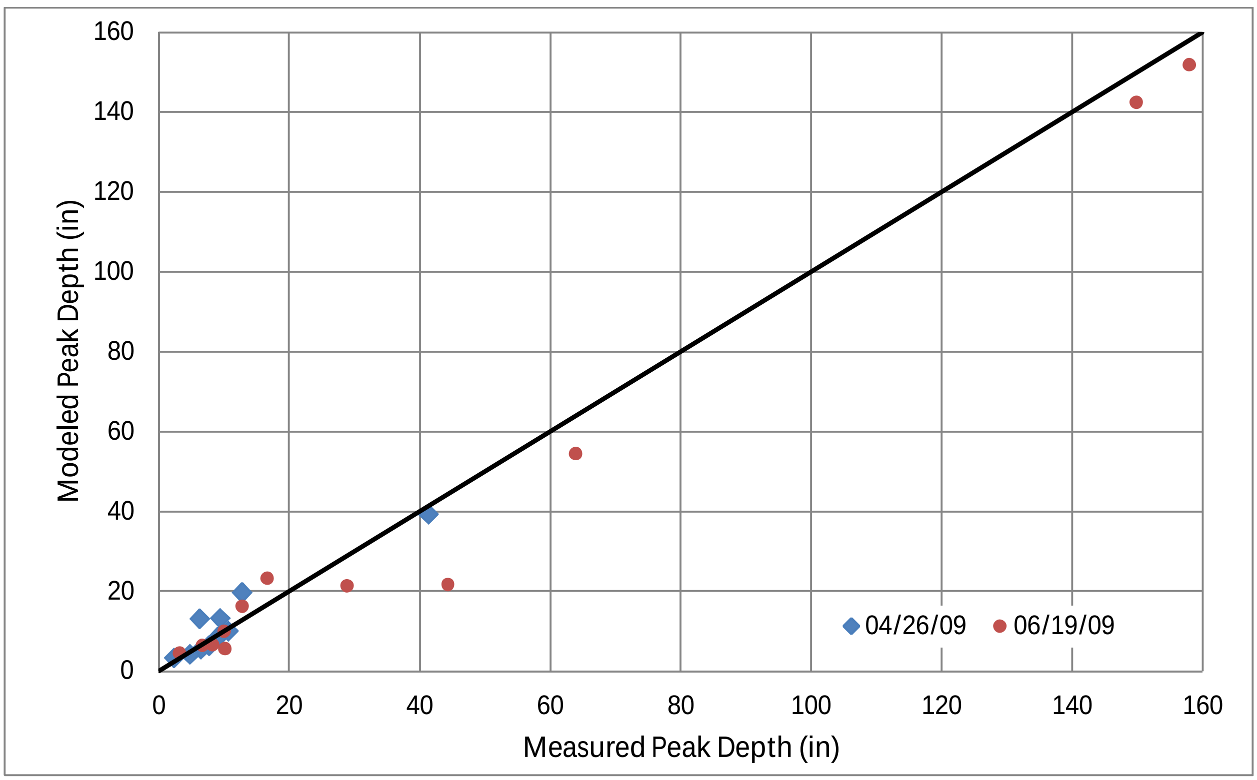


Figure 44 – Model Peak Flow Depth Calibration

6.4 PRELIMINARY HYDRAULIC EVALUATION

By evaluating the hydraulics of the collection system the model predicted for the June storm, engineers were able to identify potential system deficiencies.

6.5 ALTERNATIVE ANALYSIS

Thus far, alternative analysis has only been performed on a limited basis. This is because much of the work involved is planned for Phase II. This includes compiling projected land use data maintained by the Planning Department from which to project future flows. On the other hand, there were several projects identified for which preliminary alternative analyses were performed.

6.5.1 WEST-SIDE BYPASS

The details of the West-Side Bypass were provided in Section 3.1. The hydraulic model was modified to reflect this modification and re-run for the storm of June 2009. The resultant peak hydraulic grade line (HGL) in the proposed sewer for this event is presented in Figure 45. This HGL, produced by running the storm of June 2009, a 25-year event, resulted in some sewer surcharging but no overflow from manhole covers. The bypass could accommodate potentially greater flows, perhaps by sealing manhole covers. The sewer was sized to match the capacities of the pump stations it is intended to replace, but the 50-year and 100-year storms would likely produce flows 15% and 35% greater than the 25-year storm respectively. I&I reduction in the Pebble Valley service area could also provide in improved level of service. The sewer size could be increased to accommodate a larger storm.

Figure 46 and Figure 47 compare the Q_{Peak}/Q_d for the 25-year event for the existing sewers that form an interconnect between the Greenmeadow and Coneview pump stations. These sewers currently reach their capacity limits during a 25-year storm. However with the bypass in place, these sewers appear to have sufficient capacity available to accommodate the 100-year storm.

The bypass will transfer flow and elevate peak water levels in the 30-inch sewer into which it discharges. This sewer originates at MacArthur Road and W. St Paul Avenue and travels approximate 0.5 miles under the Fox River and to the WWTP. The 25-year HGL (Figure 48) indicates this sewer currently has capacity remaining during a 25-year storm, and can likely accommodate a 100-year storm. However it will reach it's conveyance limit (Figure 49) during the 25-year storm with the construction of the bypass. This sewer was found to contain about 6 inches of sediment and debris; cleaning could significantly increase its capacity.

6.5.2 SOUTHEAST BYPASS

The details of the Southeast Bypass were provided in Section 3.2. The hydraulic model was modified to reflect this modification and re-run for the storm of June 2009. The resultant peak HGL in the proposed sewer for this event is presented in Figure 50. The sewer is sized to match the capacities of the pump stations it replaces and will initially have excess capacity available as it has been oversized to accommodate future growth anticipated for the area that would be served by this sewer. It should have sufficient capacity to accommodate the 50-100 year storm, however this may may be diminished as future growth occurs. I&I reduction in the Heyer Drive service would free up pipe capacity.

The additional flow that would be received by the Fox Point pump station from this sewer would exceed its capacity. This station's firm capacity of 4.75 cfs would need to be increased to 19 cfs to accommodate the additional flow it would immediately receive once the bypass sewer is constructed. Its firm capacity would need to be increased to 40 cfs to match the ultimate capacity of this sewer, intended to receive flow anticipated from further development in southeast Waukesha.

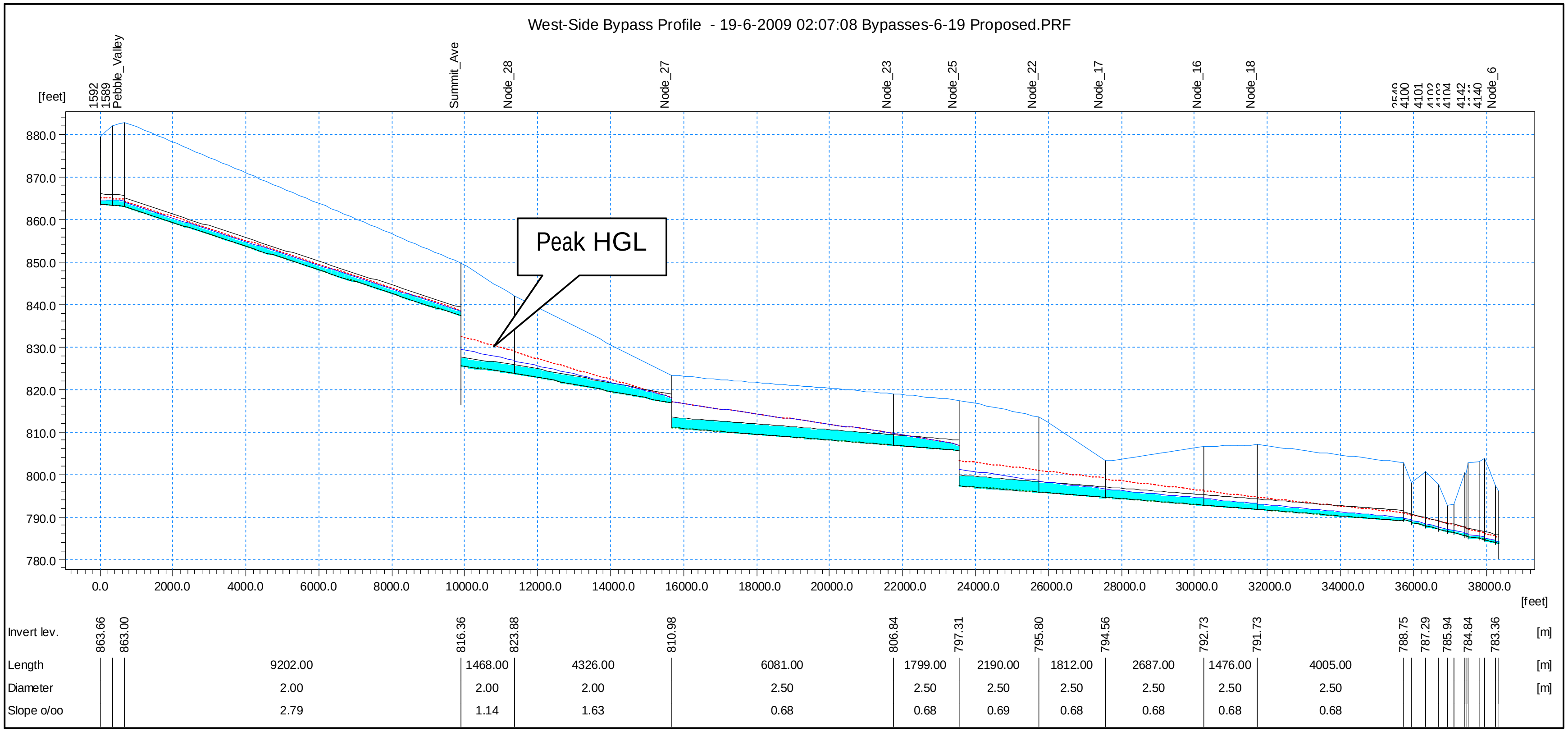


Figure 45 – West-Side Bypass Sewer HGL

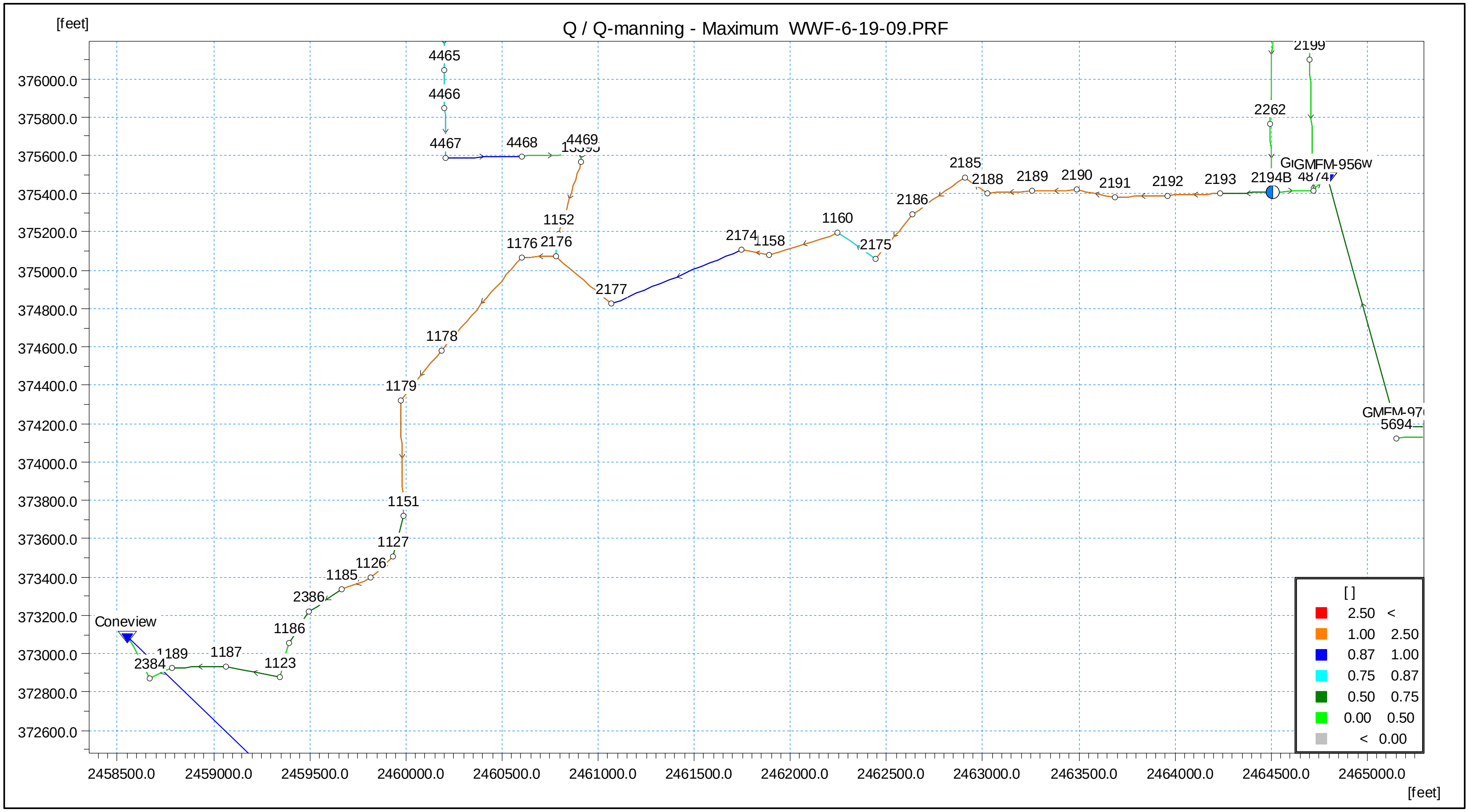


Figure 46 – 25-Year Q_{Peak}/Q_d (Existing)

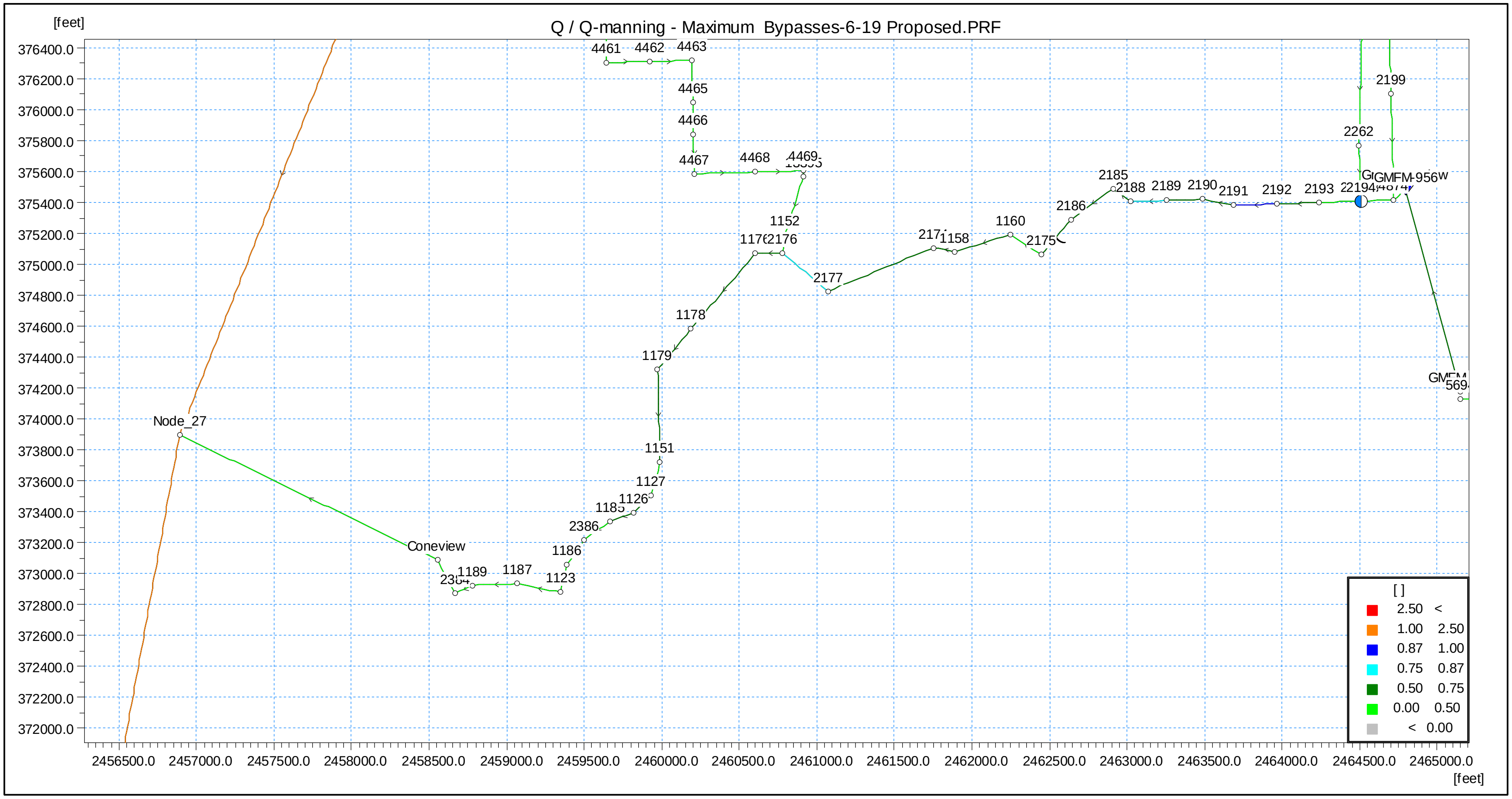


Figure 47 – 25-Year Q_{Peak}/Q_d (Proposed)

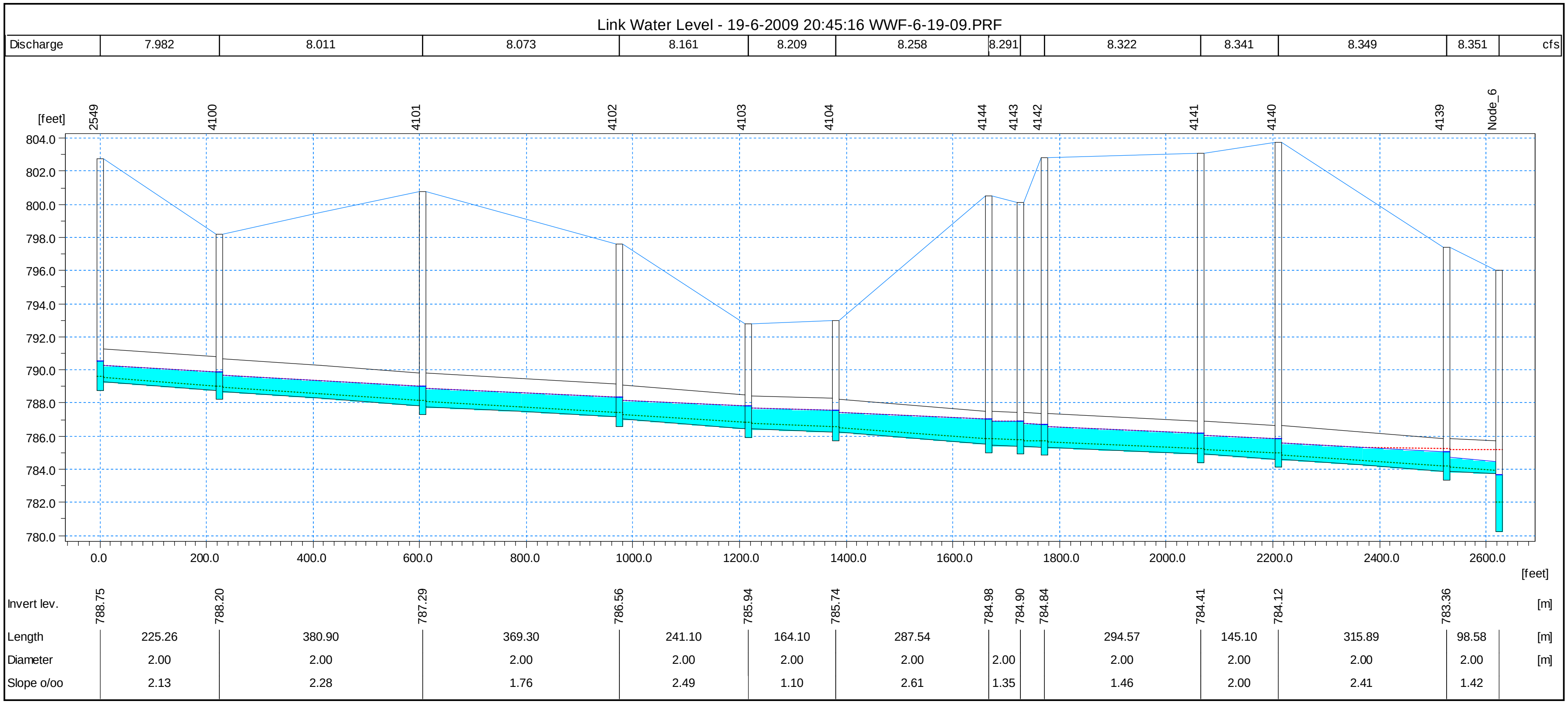


Figure 48 – 30-inch WWTP Influent Sewer 25-Yr HGL (Existing)

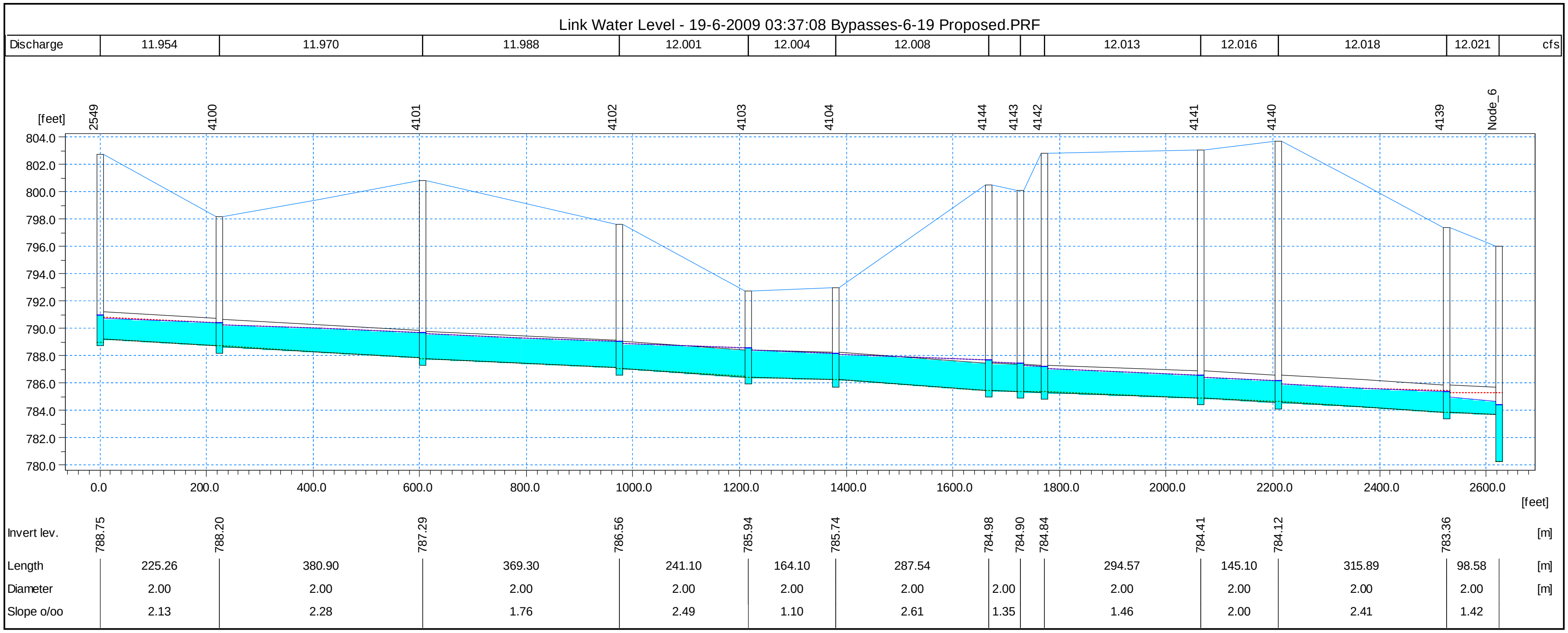


Figure 49 – 30-inch WWTP Influent Sewer 25-Yr HGL (Proposed)

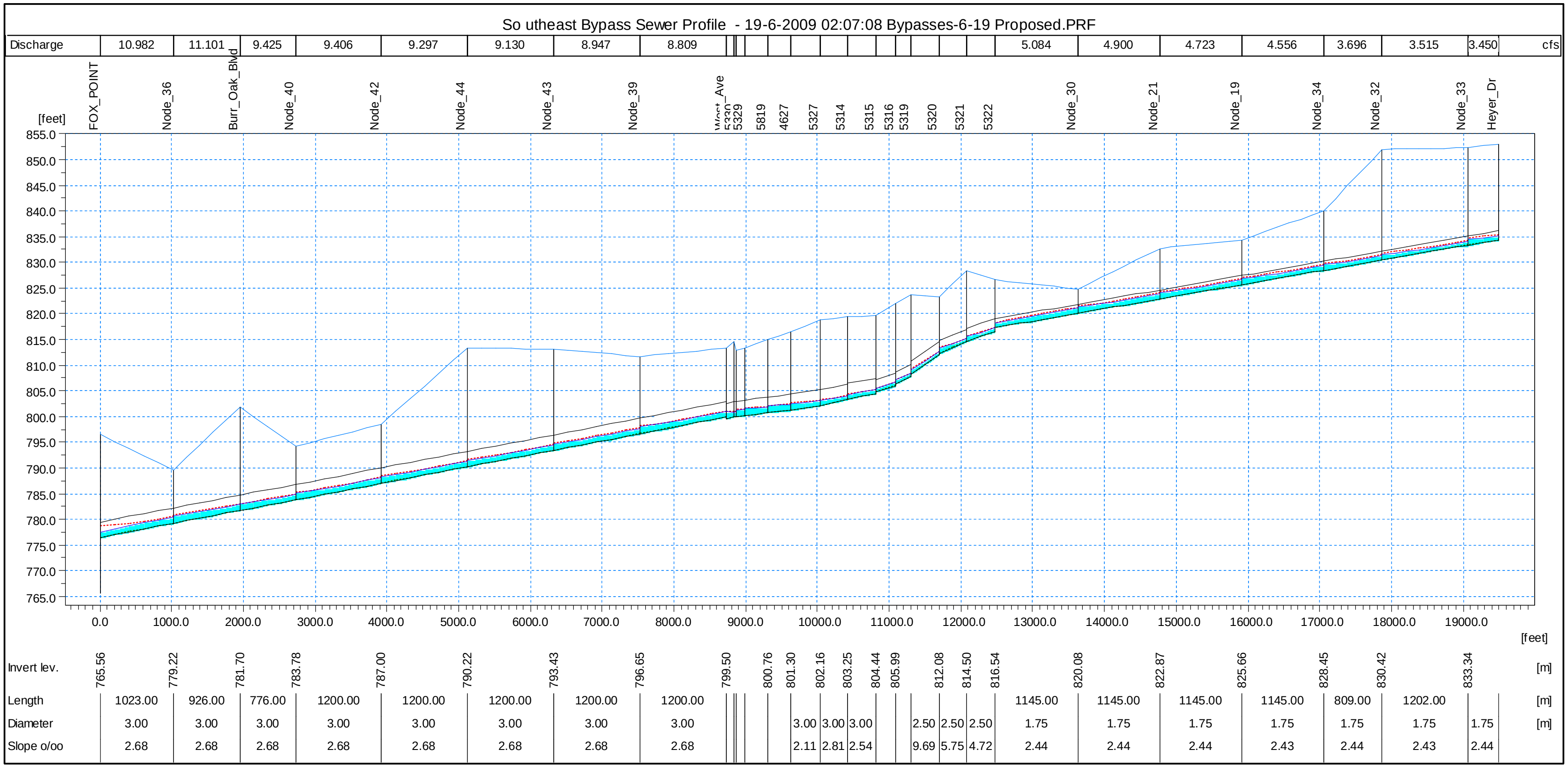


Figure 50 – Southeast Bypass Sewer 25-Year HGL

6.5.3 SENTRY AVENUE SEWER

Under current conditions, the hydraulic model indicates the 27-30-inch sewer along Sentry Ave. likely reaches capacity during the 25-year event. While the construction of the Southeast Bypass would not immediately direct additional flow into this sewer, it would likely exacerbate its overloading. This is because some of the flow currently conveyed by this sewer is attenuated by the limited capacity of the West Ave pump station. With the elimination of this pump station and the rerouting of the flow to Fox Point, peak wet weather flows would likely reach the Sentry sewer slightly faster than they do now. Figure 51 compares the ratio of the 25-year peak flow to the design capacity (Q_{Peak}/Q_d) before and after the construction of the Southeast Bypass. Figure 52 compares the resultant increase in the peak HGL.

However, there was some uncertainty as to the calibration of the model in this part of town. Therefore Donohue recommends that the City confirm the dry and wet weather response of the Sentry sewer by conducting additional flow monitoring in spring/summer.

6.5.4 MISCELLANEOUS HYDRAULIC ANALYSES

At the request of City personnel, Donohue engineers conducted several miscellaneous capacity evaluations. These are described below.

6.5.4.1 Grand Ave Sewer

During the workshop held at Waukesha Public Works on October 1, 2009, City personnel informed Donohue of their intent to rehabilitate the S. Grand Avenue sewer, and requested that Donohue evaluate whether this sewer should be upsized.

The City intends to rehabilitate the sewer along S. Grand Ave. from Estberg Ave. to W. College Ave. The model had identified this sewer as being hydraulically deficient as indicated in Figure 53. Figure 54 indicates the hydraulic grade line (HGL) that the MOUSE model predicts would have occurred during the storm of June 19, 2009. Portions of this sewer were flowing above their design capacities resulting in significant sewer surcharging, but no flooding.

This surcharging could be alleviated by upsizing 720 feet of 10” sewer to 12” and 980 feet of 12” sewer to 15” as indicated in Figure 53. The resultant HGL is presented in Figure 55. The design computations have been included as Table 13.

Upon further review, City personnel noted that even during the 100-year storm of June 2008 there were not reports of flooding along this sewer. This may be an instance where limitations in the precision of the model calibration results in output that differs from reality. Since there are no records of flood complaints to support the model predictions, City personnel have elected to rehabilitate rather than replace and upsize the Grand Avenue sewer.

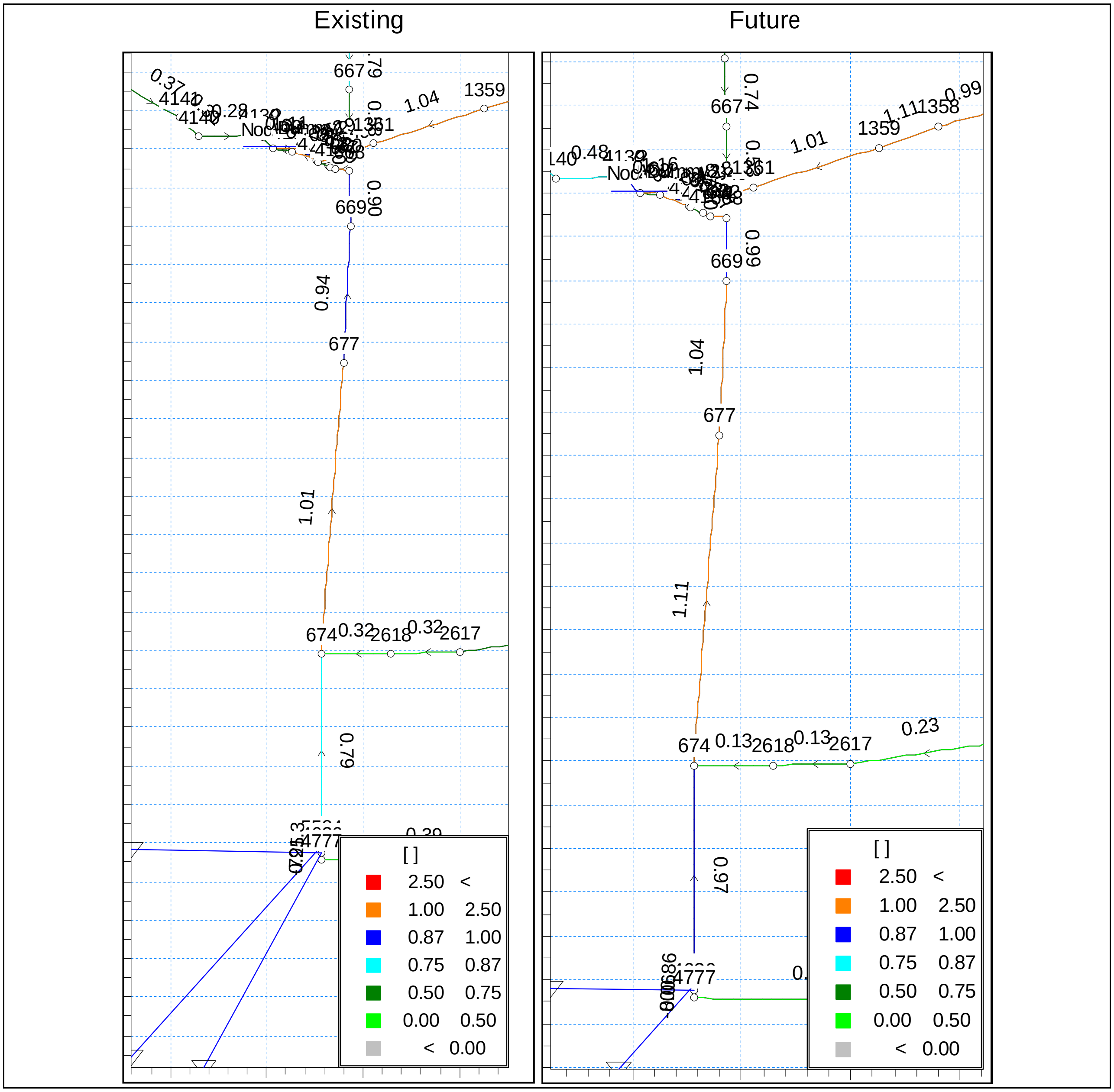
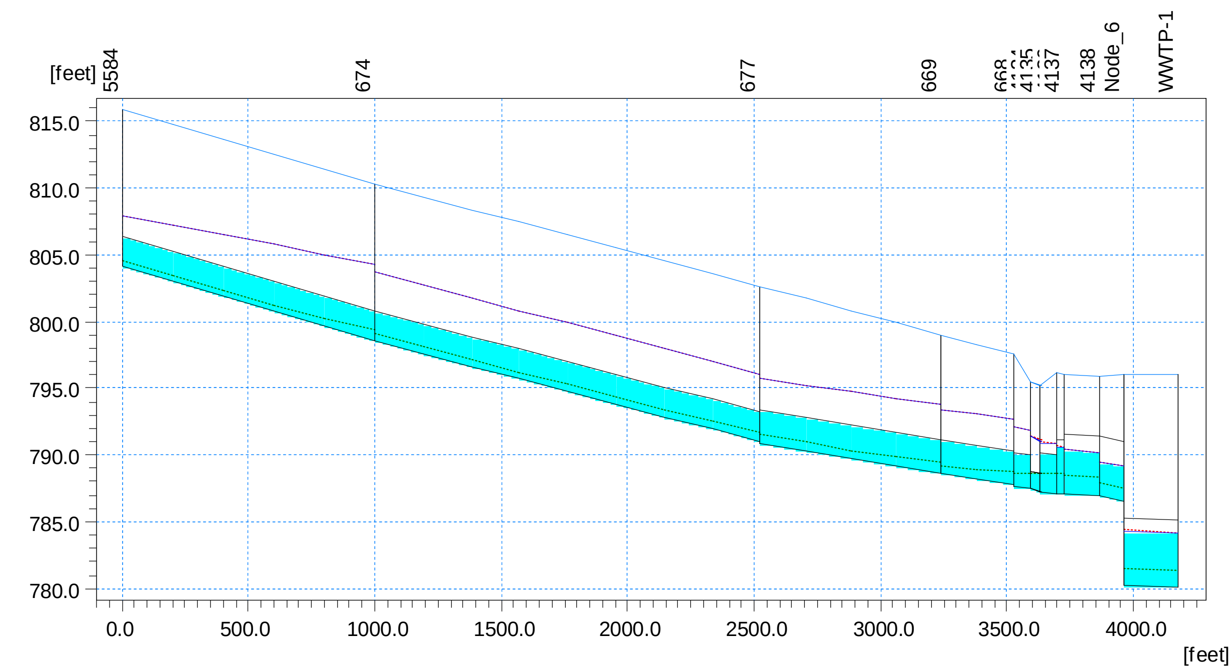


Figure 51 – Sentry Sewer 25-Year Q_{Peak}/Q_d



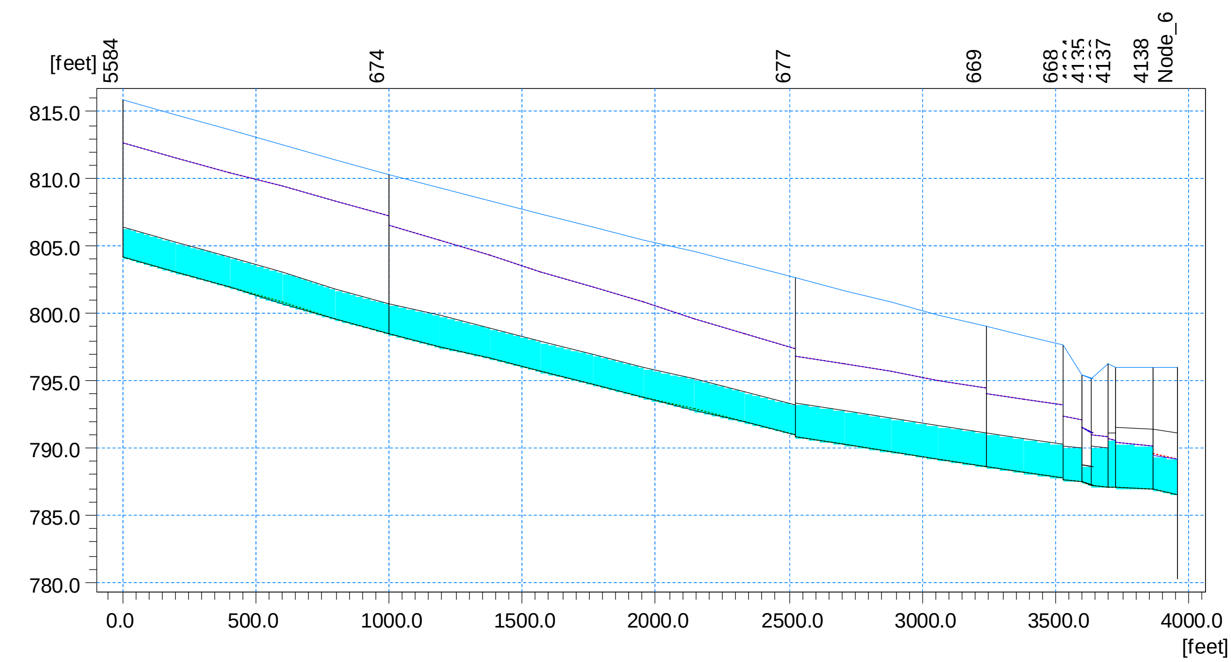


Figure 52 – Sentry Sewer 25-Yr HGL

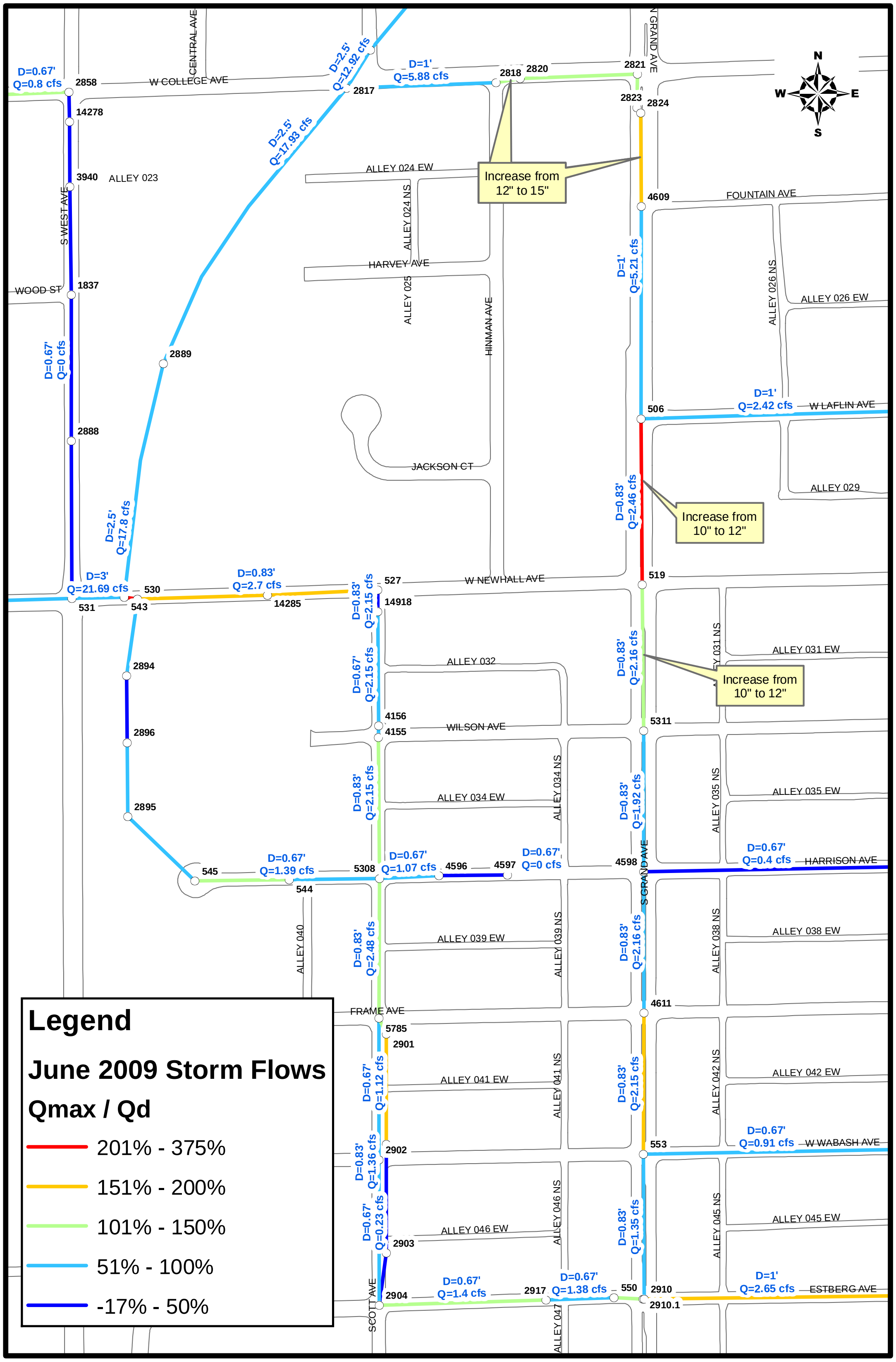


Figure 53 – Grand Avenue Sewer Capacities

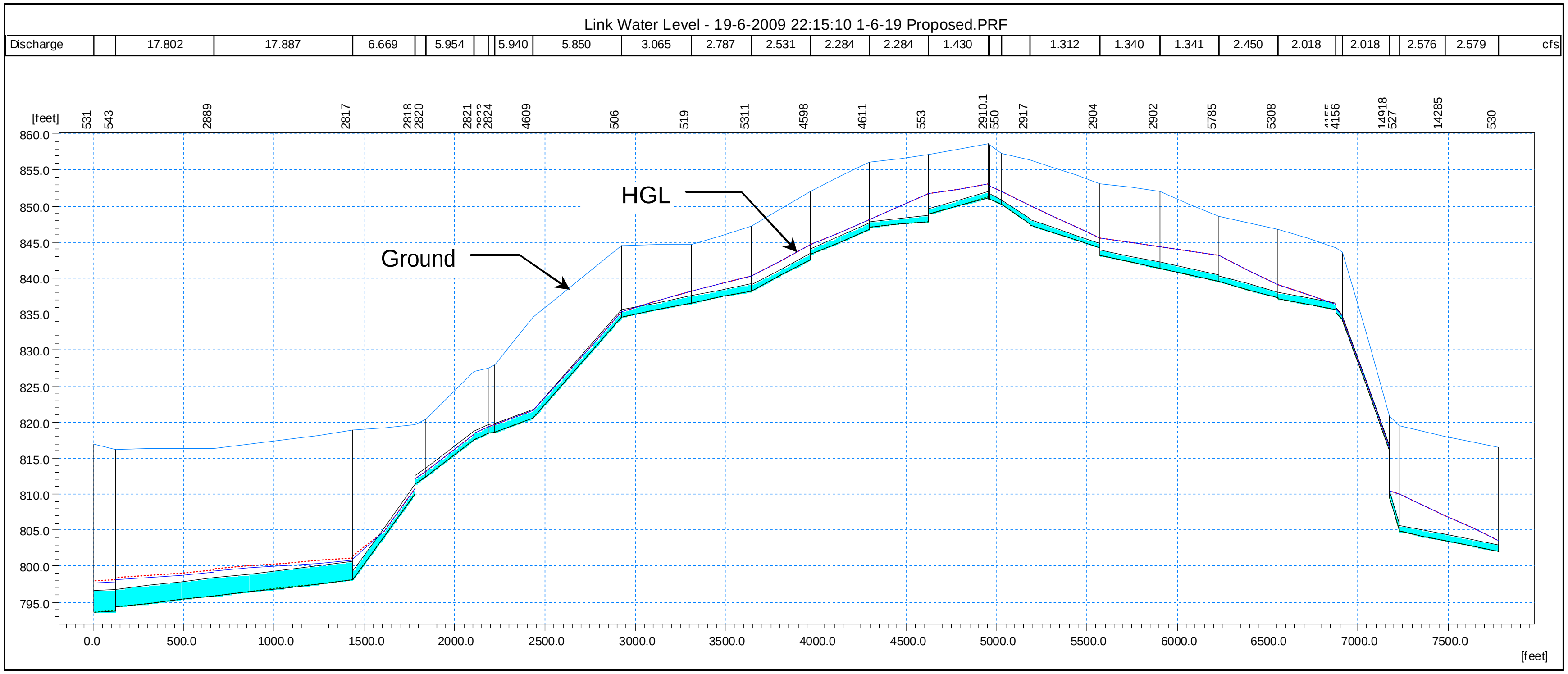


Figure 54 – Grand Avenue Sewer HGL (Existing)

Table 13 – Grand Avenue Sewer Design Computations

			Existing Sewer										Proposed Sewer									
Pipe ID	Upstream MH	Downstream MH	UpLevel	DwLevel	Length	Slope (%)	Dia (in)	Material	Manning	Qmax (cfs)	Qd (cfs)	Q_{max}/Q_d	UpLevel	DwLevel	Length	Slope (%)	Dia (in)	Material	Manning	Qmax (cfs)	Qd (cfs)	Q_{max}/Q_d
662	2910	553	851.22	848.86	335	0.706	10	Concrete	0.013	1.35	1.84	73%	851.22	848.86	335	0.71	10	Concrete	0.013	1.35	1.84	73%
663	553	4611	847.86	847.06	326	0.245	10	Concrete	0.013	2.15	1.09	198%	847.86	847.06	326	0.25	10	Concrete	0.013	2.15	1.09	198%
666	4611	4598	846.79	843.26	325	1.085	10	Concrete	0.013	2.16	2.29	95%	846.79	843.26	325	1.08	10	Concrete	0.013	2.16	2.29	95%
665	4598	5311	842.56	838.23	326	1.329	10	Concrete	0.013	1.92	2.53	76%	842.56	838.23	326	1.33	10	Concrete	0.013	1.92	2.53	76%
667	5311	519	838.23	836.56	337	0.495	10	Concrete	0.013	2.16	1.54	140%	838.23	836.56	337	0.50	12	Concrete	0.013	2.16	2.51	86%
669	519	506	836.56	835.56	382	0.262	10	Concrete	0.013	2.46	1.12	219%	836.56	834.56	382	0.52	12	Concrete	0.013	2.46	2.58	95%
668	506	4609	834.56	820.56	491	2.852	12	Concrete	0.013	5.21	6.03	86%	834.56	820.56	491	2.85	12	Concrete	0.013	5.21	6.03	86%
670	4609	2824	820.56	818.56	215	0.929	12	Concrete	0.013	5.21	3.44	151%	820.56	818.56	215	0.93	15	Concrete	0.013	5.21	6.24	84%
671	2824	2823	818.56	818.39	15	1.102	12	Concrete	0.013	5.21	3.75	139%	818.56	818.39	15	1.10	15	Concrete	0.013	5.21	6.79	77%
672	2823	2821	818.39	817.51	77	1.143	12	Concrete	0.013	5.21	3.82	137%	818.39	817.51	77	1.14	15	Concrete	0.013	5.21	6.92	75%
673	2821	2820	817.51	812.41	271	1.882	12	Concrete	0.013	5.21	4.90	106%	817.51	812.41	271	1.88	15	Concrete	0.013	5.21	8.88	59%
35	2820	2818	812.41	811.34	57	1.894	12	Concrete	0.013	5.21	4.91	106%	812.41	811.34	57	1.89	15	Concrete	0.013	5.21	8.91	58%
36	2818	2817	810.06	798.10	344	3.476	12	Concrete	0.013	5.88	6.65	88%	810.06	798.10	344	3.48	15	Concrete	0.013	5.88	12.06	49%

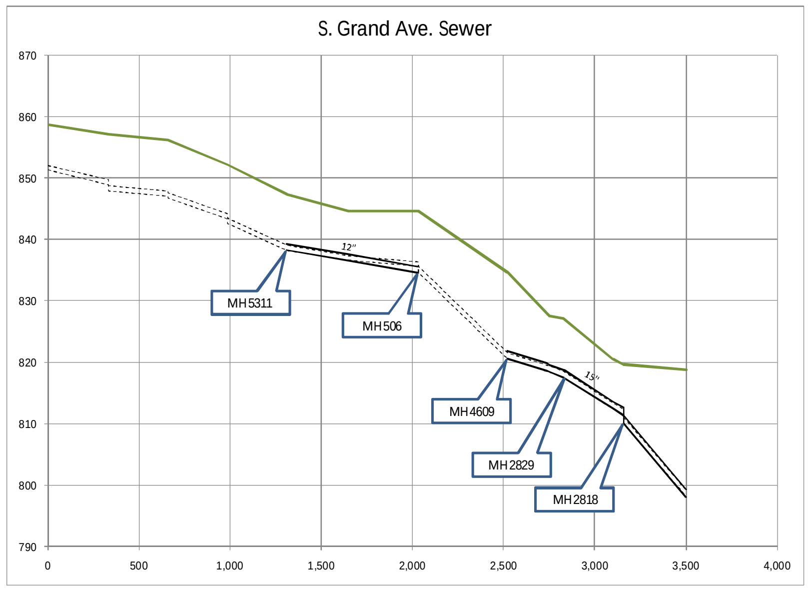


Figure 55 – Grand Avenue Sewer Profile (Proposed)

6.5.4.2 Sunset Drive Sewers

There are parallel 12-inch and 18-inch sewers along Sunset Drive from S West Avenue to S Grand Avenue. These were found to be in poor condition. City personnel intended to either replace the 18-inch sewer or rehabilitate it using cured-in-place-pipe (CIPP) and wanted to know whether the 12-inch sewer should be rehabilitated or abandoned.

Both of these sewers were included in the collection system model. Using model, it was determined that the rehabilitated 18-inch sewer would have adequate capacity to accommodate existing flows. Once the Southeast Bypass is constructed, much of the flow that is currently conveyed by this sewer will be redirected away from this sewer, providing even greater available capacity. Therefore Donohue recommends that the City rehabilitate the 18-inch sewer and abandon the 12-inch sewer.

6.5.4.3 University Drive Sewer

The sewer along University Drive from Sunkist Avenue to Summit Ave (Figure 56) was found to be in poor condition. City personnel proposed sliplining this sewer. The design capacity of this sewer (15 cfs), is much greater than that of the pump station. Therefore it can be safely sliplined without negatively impacting conveyance.

It appears likely that if the West-Side Bypass were constructed and the Pebble Valley and Tallgrass pump stations eliminated, the only flow remaining in this sewer would be from the University of Wisconsin at Waukesha and a few other adjacent properties.

6.6 WWTP FLOW PEAK FLOW EVALUATION

All flows conveyed by the collection system must ultimately be treated at the WWTP. Alleviating a hydraulic deficiency has the potential to put additional strain on the plant. Therefore one must consider what impact any collection system modifications are likely to have on peak plant flows.

The plant is currently rated for a Qavg of 16 MGD; a review of four years of plant flow data indicated an actual Qavg of 9.8 MGD and a Qmax of 48.7 MGD, a peaking factor of 5. The City is required to provide complete treatment of a 25-year storm and primary treatment of a 100-year storm. To our knowledge, there has been only one plant bypass as a result of excess flow, and this was during the storm of June 2008, a 100-year event. Table 14 lists the likelihood of recurrence of a range of peak flows to the plant.

Table 14 – WWTP Peak Flow Frequencies

Recurrence Interval (years)	% of 25-Year Storm*	Qp (MGD)	Probability of Occurrence
25	100%	55	0.011%
50	115%	63	0.005%
100	135%	85	0.003%

* Ratio of rainfall volumes from Bulletin 71 (Midwestern Climate Center, 1992)

None of the collection system improvements considered thus far are likely to significantly increase plant flows. The model indicates the two major modifications under consideration, the West-Side and Southest Bypasses, would increase the 25-year peak flow reaching the plant from 54.3 MGD to 55.5 MGD, a 2% increase. This is a brief, instantaneous peak, beyond the plant's current capacity, and would be stored briefly in the influent sewers. Additional flows as a result of development will be considered during Phase II Master Planning.

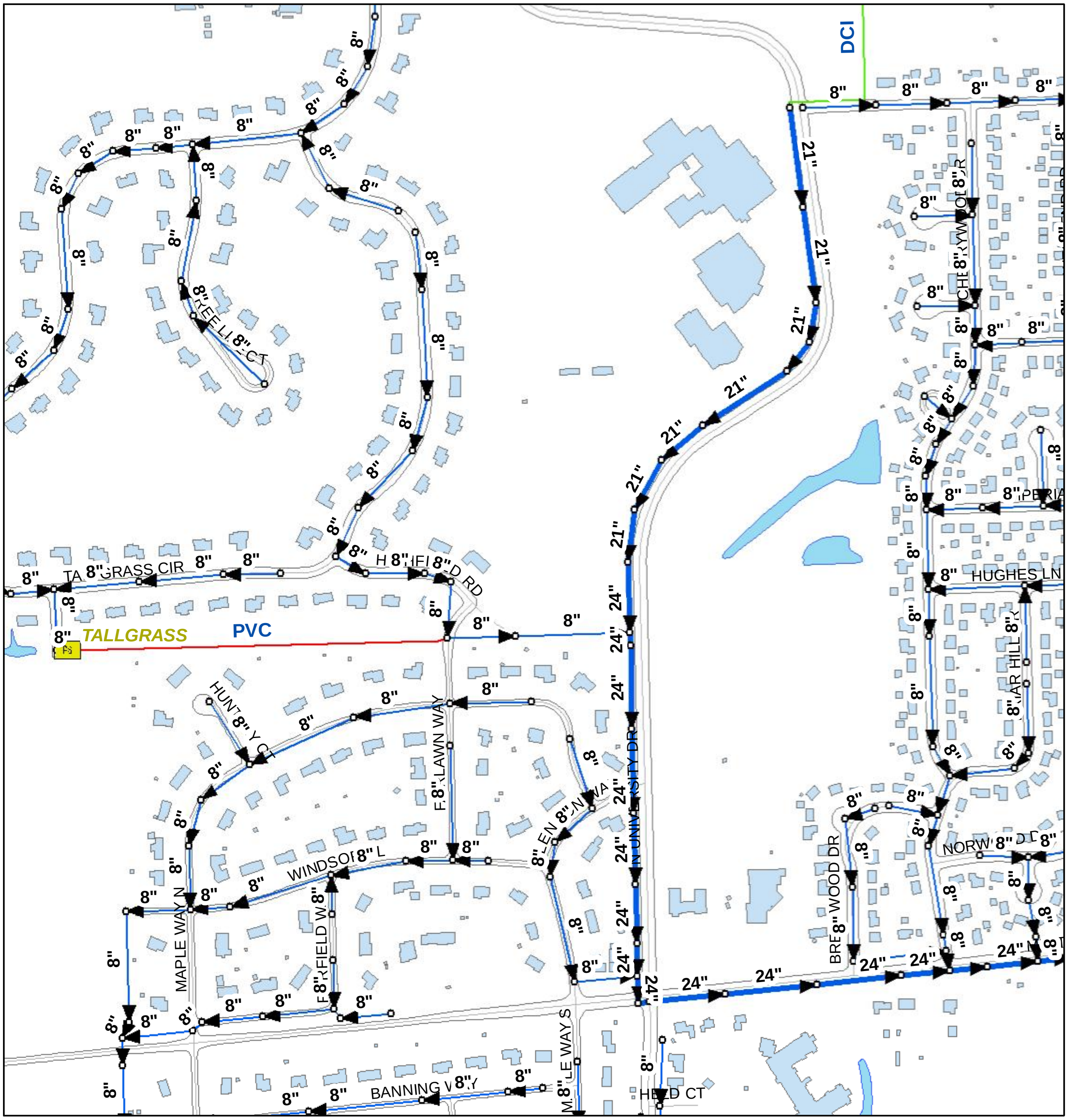


Figure 56 – University Drive Sewer

CHAPTER VII – CAPACITY, MAINTENANCE, OPERATIONS, & MAINTENANCE (CMOM) PROGRAM

7.1 FORCE MAIN DESKTOP RISK ASSESSMENT

A force main risk assessment was completed for the City to analyze the likelihood of future leaks and/or failures of force mains and their consequences. Risk assessment methodology, documented by several Water Environment Research Foundation and Water Research Foundation papers, was used to complete the analysis. This assessment provides "a logical and systematic means for determining the priorities for subsequent inspections and the eventual rehabilitation of sewers" (Zhao, McDonald, & Kleiner, 2001). It is important to prioritize force mains for inspection and rehabilitation so as to prepare for the future on a limited budget. This will enable The City to maintain their force mains proactively rather than reactively, which will improve system reliability and enable better forecasting of maintenance budgets.

In general, condition assessments follow a series of steps as shown in Figure 57; the scope of this phase involved prioritizing force mains for inspection using a qualitative risk assessment. Phase II of this project will assess the actual physical condition of the force mains that are ranked highest on the priority list.

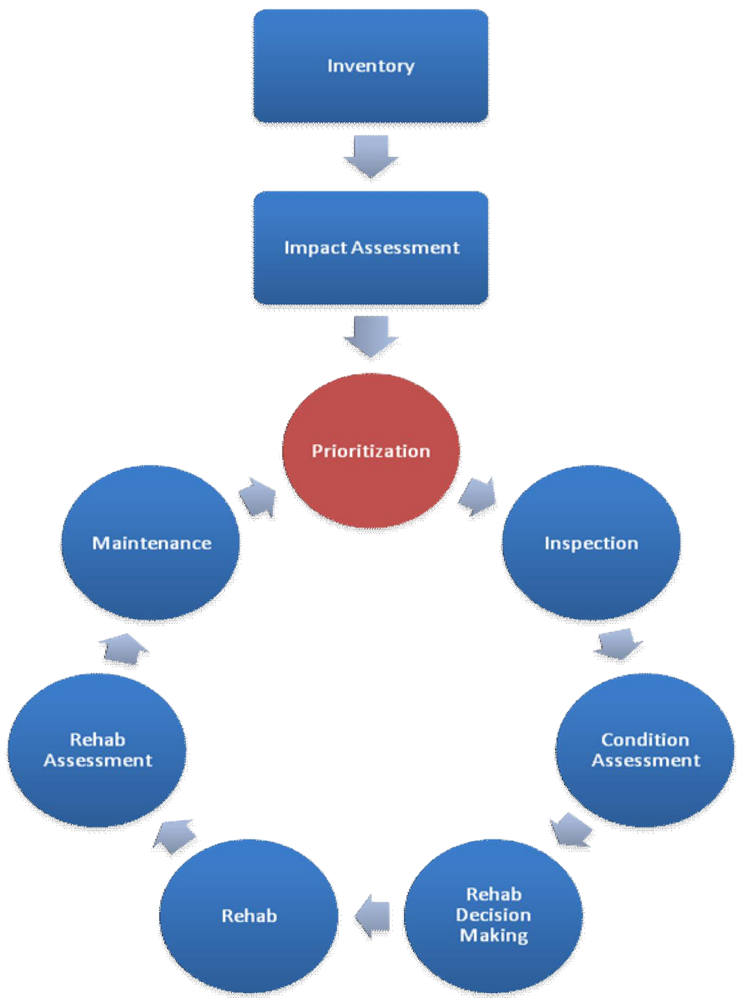


Figure 57 - Condition Assessment Process

Adapted from: (Zhao, McDonald, & Kleiner, 2001)

7.1.1 RISK ASSESSMENT METHODOLOGY

Risk—as discussed in *Guidelines for Condition Assessment and Rehabilitation of Large Sewers,* (Zhao, McDonald, & Kleiner, 2001) and *Condition Assessment of Wastewater Collection Systems* (Feeney, Thayer, Bonomo, & Martel, 2009)—is composed of two elements, severity and probability, and the definition of risk is the product of the two (Fact Sheet on Probabilistic Risk Assessment, 2010). In this case, the risk of a force main failure, relative to all other force mains in the system, was analyzed based on the likelihood of failure and the consequence of its failure. Since a true probability could not be determined without actual field testing, a numerical scaling system was used to rank each criterion, with five having the largest impact on either likelihood or consequence and one having the least. Based on the product of likelihood and consequence, the force mains were ranked relative to each other.

Figure 58 (Thomson & Wang, 2009) illustrates the relationship between likelihood and consequence. Force mains that have both a high likelihood of failure and a severe consequence of failure rise to the top of the priority list. It is important to note that this ranking is purely qualitative and the force mains could only be assessed relative to one another. Therefore, a high ranking does not necessarily indicate imminent failure; it simply suggests that the highest ranking force main is more likely to fail before the others or has a greater consequence of failure, based on the rating criteria.

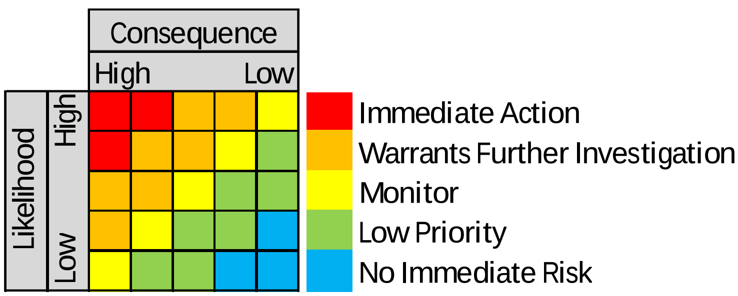


Figure 58 - Risk Assessment Matrix

Source: (Thomson & Wang, 2009)

7.1.2 RATING CRITERIA

7.1.2.1 Consequence of Failure

Consequence of Failure factors were based on socio-economic and environmental considerations; they refer to the impact on the environment and the public should a force main fail (Feeney, Thayer, Bonomo, & Martel, 2009). Consequence is a function of the location of the force main and the nature of the surrounding area. Consequence data was gathered from aerial and GIS maps of the City.

7.1.2.1.1 Area Type

Area type refers to the risk of public exposure to raw sewage should a force main leak or break (Thomson, et al., 2004). The risk of this is highest in a residential neighborhood and lowest in areas that are undeveloped.

Table 15 depicts the rating criteria for area type. To account for multiple area types, each rating was multiplied by the percentage of area type.

Table 15 - Area Type Consequence Ratings

Area Type	Rating
Residential	5
Commercial	4
Industrial	3
Waterway	2
Undeveloped	1

7.1.2.1.2 Street Type

Since sewers generally follow roadways, one impact of a force main failure is traffic disruption (Zhao, McDonald, & Kleiner, 2001). Major roadways with higher traffic flow have greater consequences in the event of a force main failure. Table 16 shows the rating criteria for street type; "none" applies to force mains traversing areas without roadways. To account for multiple street types, each rating was multiplied by the percentage of street type in that area.

Table 16 - Street Type Consequence Ratings

Street Type	Rating
Highway	5
Main Road	4
Local	3
Private	2
None	1

7.1.2.1.3 Pipeline Location

Pipeline location refers to the proximity of a force main to surface water (Feeney, Thayer, Bonomo, & Martel, 2009). This is the primary measure being used to identify environmental consequences of force main failure. Two routes of flow to surface water are considered—directly overland to the waterway or indirectly by way of storm sewers. A third situation, possible in undeveloped areas, is "no storm sewer present". In this case, sewage would not be expected to reach a waterway. Table 17 shows the consequence ratings for each scenario.

Table 17 - Pipeline Location Consequence Ratings

Pipeline Location	Rating
Surface water present	5
Storm sewer present	3
No storm sewer present	1

7.1.2.1.4 Size of Main

The force mains under consideration range in size from 4 inches to 16 inches. Force main size affects the degree of consequence for the surrounding area—larger mains having a more severe consequence, since a larger flow is expected in these mains (Thomson & Wang, 2009). Mains were rated on a scale from one to five, with 16-inch mains rated highest and 4-inch mains rated lowest.

7.1.2.1.5 Population Equivalence

The population equivalent (PE) serves to represent the magnitude of the population that will be inconvenienced by a force main failure; it factors in the affect of large users on the system, such as industry. One population equivalent is 54 gallons of sewage per day. PE was normalized on a scale from one to five so as to be consistent with the other rating criteria.

7.1.2.2 Likelihood of Failure

The Likelihood of Failure criteria were developed based on the most common causes of failure of force mains in the United States. Figure 59 depicts these failure modes (Jason Consultants, LLC, 2007).

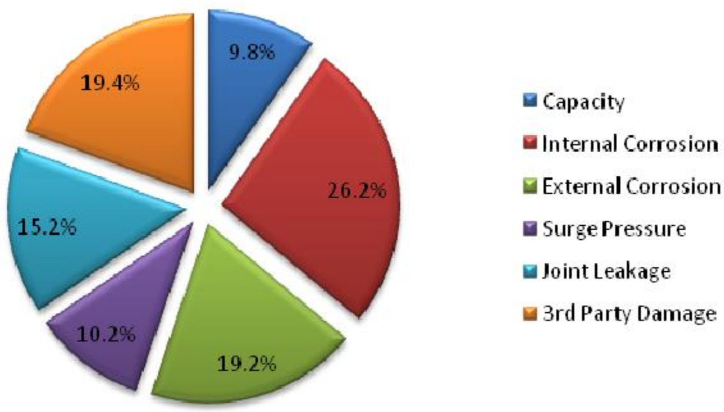


Figure 59 - Most Common Causes of Force Main Failures

Source: (Jason Consultants, LLC, 2007)

Our criteria are based on joint leakage (leaks per unit length), external corrosion (corrosive environment), surge pressure, capacity (operating pressure), and one more factor—remaining life. The reasoning for this addition is noted below; however, internal corrosion and 3rd party damage, while significant contributors to force main failure, were not included. This is because internal corrosion is hard to predict at a "desktop" level as it is dependent on the nature of the chemicals and sewage running through the pipe. 3rd party damage is not a predictable factor and therefore cannot be included.

It is also important to note that many of the above criteria are functions of age (leakage, internal and external corrosion) and that the affect of one can exacerbate the affect of another. For instance, a force main may not fail due to capacity alone; however, if a force main is already largely corroded internally and externally, operating pressure may exceed what the force main can actually handle a failure might occur. To account for this while remaining consistent in the analysis, the average of rating was used for the final likelihood rating. In that way, if a force main is in a highly corrosive soil *and* has high operating pressures, the overall average will be larger than that of others.

7.1.2.2.1 Leaks per Unit Length

Leaks per unit length takes into consideration failures that have already occurred and therefore could indicate future failures in that force main. The rating factor was calculated by the number of documented leaks on the force main divided by the length of the force main in miles, then normalized on a scale from one to five, with force mains with the most leaks per unit length receiving the highest rating. Force mains without a documented leak were assigned a value of one.

7.1.2.2.2 Remaining Life

Pipe age is not necessarily a predictor of failure, "but [an] important factor in the nature and likelihood of failure" (Thomson & Wang, 2009). Rather than considering age in and of itself, we considered the remaining useful life of the force main. In this way, we can account for force mains that may have a low ranking for other factors, but may only have relatively few years of useful life remaining and therefore warrant a physical condition assessment sooner rather than later.

Data on the life expectancy of force mains varies widely by material and installation date. Typical, conservative estimates for life expectancy were used. There is considerably more data available for force mains made of ferrous materials (cast iron and ductile iron) than those made of plastics (PVC and HDPE), since they have been in use much longer and account for approximately 60% of the force mains throughout the United States (Jason Consultants, LLC, 2007).

Cast Iron: The Cast Iron Soil Pipe Institute (Cast Iron Soil Pipe Institute - CISPI, 2010) claims "The oldest installations of cast iron pipe are in underground lines... Many are over 100 years old." However, UK Water Industry Research found that average life of ferrous mains was highly variable, some lasting as little as 18 years, others lasting as long as 150 years. To be conservative, an intermediate number of 60 years was chosen as an average life span of cast iron pipe.

Ductile Iron: The Ductile Iron Pipe Research Association (Ductile Iron Pipe Research Association - DIPRA, 2010) says, "Properly designed and installed Ductile Iron pipe systems could easily have a life expectancy of more than 100 years. As with cast iron, the average life expectancy of ductile iron is highly variable, and an intermediate value of 60 years was chosen for the life span. Also, since ductile iron is essentially a subset of cast iron (American Ductile Iron Pipe - ACIPCO, 2010), the expected life should be the same as cast iron.

PVC: The Uni-Bell PVC Pipe Association (Uni-Bell PVC Pipe Association, 2010) "consider[s] one hundred years an extremely conservative estimate for the service life of a properly designed and installed PVC pipe." Considering PVC has not been in use long enough to observe actual service life, and since failure rates of PVC pipe are considerably less than ferrous pipes (Cook, McAndrew, & Shuker, 2009), a life expectancy of 100 was assigned to PVC force mains.

HDPE: There is even less available data on HDPE pipe than PVC; however, we assumed an equal value of 100 years for the life expectancy of HDPE pipes. Like PVC, HDPE is not susceptible to corrosion and can therefore be expected to last considerably longer than ferrous materials.

Lastly, 50 years was used for any force mains for which the material was unknown.

For all pipes, the remaining life was determined by subtracting the age from the life expectancy. These values were then normalized on a scale from one to five to determine a rated value for this analysis.

7.1.2.2.3 Corrosive Environment

The United States Department of Agriculture (USDA) maintains a website, Web Soil Survey, containing data on soils throughout the US. By selecting an "area of interest", we were able to determine the corrosivity of the soils in the vicinity of each force main (see Figure 60).

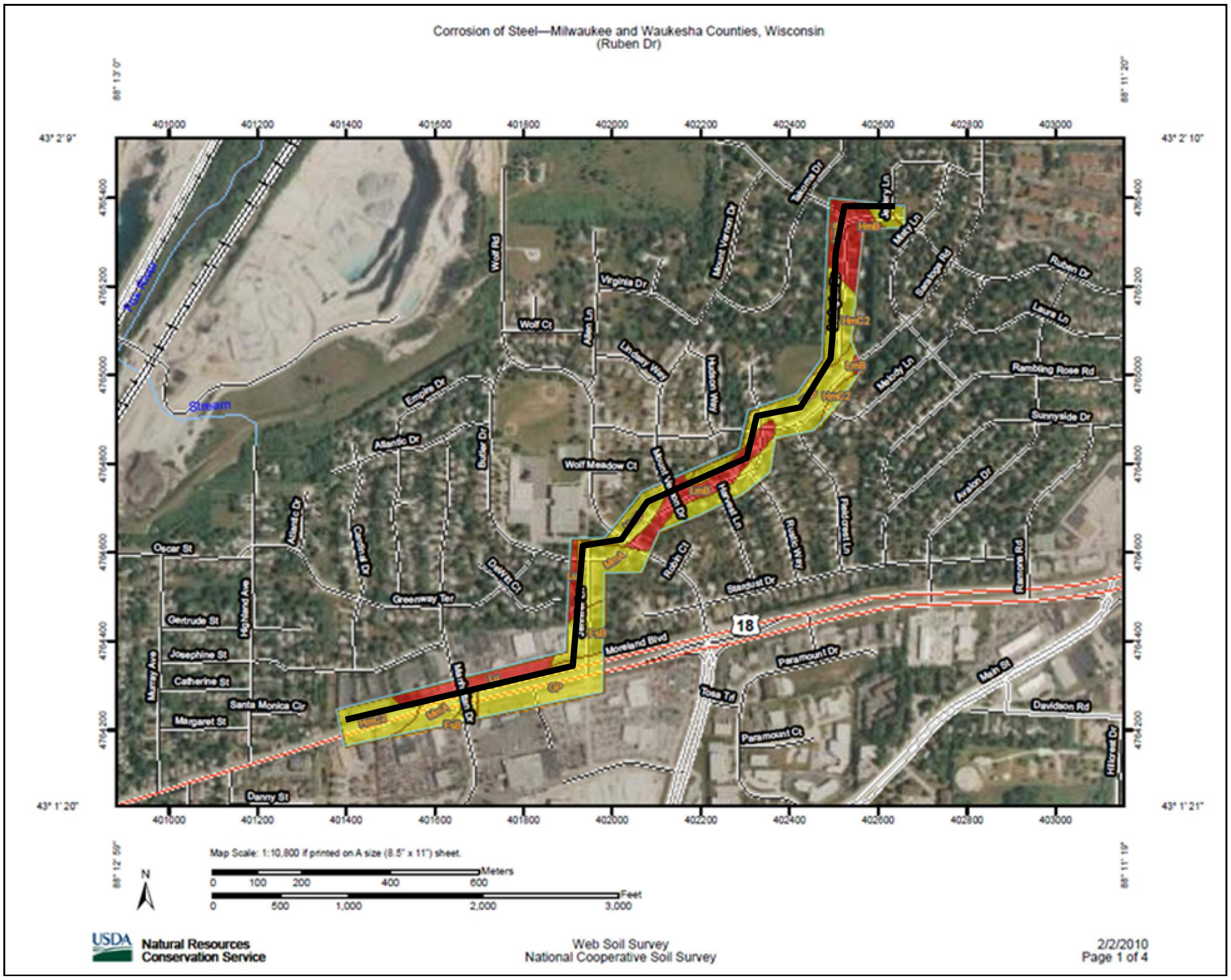


Figure 60 - USDA Area of Interest Soil Survey for the Ruben Drive Force Main

Source: (USDA, 2010)

The USDA then rates the soils in the following way:

"Risk of corrosion" pertains to potential soil-induced electrochemical or chemical action that corrodes or weakens uncoated steel. The rate of corrosion of uncoated steel is related to such factors as soil moisture, particle-size distribution, acidity, and electrical conductivity of the soil. Special site examination and design may be needed if the combination of factors results in a severe hazard of corrosion.

The risk of corrosion is expressed as 'low,' 'moderate,' or 'high.'" (USDA, 2010).

For the example in Figure 60, the Ruben Drive Force Main, the data is summarized in Table 18.

Table 18 - Soil Corrosivity Data for the Ruben Drive Force Main

Corrosion of Steel— Summary by Map Unit — Milwaukee and Waukesha Counties, Wisconsin				
Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
FsB	Fox silt loam, 2 to 6 percent slopes	Moderate	3.0	6.8%
GP	Gravel pit	Moderate	5.5	12.6%
HmB	Hochheim loam, 2 to 6 percent slopes	Moderate	2.9	6.5%
HmC2	Hochheim loam, 6 to 12 percent slopes, eroded	Moderate	7.3	10.0%
LmB	Lamartine silt loam, 1 to 4 percent slopes	High	8.1	18.4%
Lu	Loamy land	High	3.7	8.3%
MmA	Matherton silt loam, 1 to 3 percent slopes	Moderate	5.3	12.1%
ThB	Theresa silt loam, 2 to 6 percent slopes	Moderate	8.3	18.8%
Totals for Area of Interest			44.1	100.0%

To obtain a rating for each force main, areas with "high risk of corrosion" were assigned a value of five, areas with "moderate risk of corrosion" were assigned a value of three, and areas with "low risk of corrosion" were assigned a value of one. To account for multiple soil types, each rating was multiplied by the percentage of soil in that area. For example, Ruben Drive's corrosion rating was calculated as follows:

(18.4%+8.3%)*5 + (6.8%+12.6%+6.5%+16.6%+12.1%+18.8%)*3 = 3.5

The same method was applied to all ferrous metal force mains. Soils were not examined in areas where plastic pipes are used; these force mains are given a value of zero to completely eliminate the effects of corrosion. Soil corrosion reports for all ferrous force mains are included in the Appendix.

7.1.2.2.4 Operating Pressure

Pipe capacity was evaluated based on the ratio of actual operating pressure to its pressure rating. A higher ratio indicates that a force main is operating near design point and is more likely to fail. The pressure rating of each pipe is largely a function of pipe material, diameter, and wall thickness, as shown in Table 19. Since wall thickness could not be determined, the most conservative value was used for each force main.

<u>Pressure Capacity</u>
Ductile Iron/Cast Iron: The American Cast Iron Pipe Company (American Ductile Iron Pipe - ACIPCO, 2010) provides the following information for cast iron ductile iron pipe, which is based on the ANSI/AWWA C150/A21.50 standard:

Table 19 - Ductile Iron Pipe Pressure Ratings

Size in.	Outside Diameter in.	Pressure Class				
		150	200	250	300	350
		Nominal Thickness in inches				
4	4.8	-	-	-	-	0.25
6	6.9	-	-	-	-	0.25
8	9.05	-	-	-	-	0.25
10	11.1	-	-	-	-	0.26
12	13.2	-	-	-	-	0.28
14	15.3	-	-	0.28	0.3	0.31
16	17.4	-	-	0.3	0.32	0.34
18	19.5	-	-	0.31	0.34	0.36
20	21.6	-	-	0.33	0.36	0.38
24	25.8	-	0.33	0.37	0.4	0.43
30	32	0.34	0.38	0.42	0.45	0.49
36	38.3	0.38	0.42	0.47	0.51	0.56
42	44.5	0.41	0.47	0.52	0.57	0.63
48	50.8	0.46	0.52	0.58	0.64	0.7
54	57.56	0.51	0.58	0.65	0.72	0.79
60	61.61	0.54	0.61	0.68	0.76	0.83
64	65.67	0.56	0.64	0.72	0.8	0.87

Source: (American Ductile Iron Pipe - ACIPCO, 2010)

> "Pressure classes are defined as the rated water working pressure of the pipe in psi. The thicknesses shown are adequate for the rated water working pressure plus a surge allowance of 100 psi. Calculations result in net thicknesses and are based on a minimum yield strength in tension of 42,000 psi and 2.0 safety factor times the sum of working pressure and 100 psi surge allowance." (American Ductile Iron Pipe - ACIPCO, 2010)

PVC/HDPE: The Uni-Bell PVC Pipe Association (Uni-Bell PVC Pipe Association, 2010) uses the AWWA C900 standard, which states that "a 'Pressure Class' design approach [is] based on a 2.0 safety factor. AWWA C900 is intended for use inside the "looped" perimeter of an urban water system where piping system geometry is complex. Surge pressures should be accounted for in the design." This standard is used for pipes in water systems with diameters ranging from 4 to 12 inches. The next class, AWWA C905, which accounts for diameters ranging from 12 to 48 inches, has similar pressure ratings. The pressure rating varies depending on the pipe product; since we are unable to determine the exact plastic product used, the most conservative value of 100 psi was used for a pressure rating on plastic pipes (Uni-Bell PVC Pipe Association, 2010).

7.1.2.2.5 Surge Pressure

Even more than high operating pressures, surge can cause a force main failure. A sudden change in pressure will often cause a rupture in corrosion-weakened pipe walls. A surge analysis was performed on a select number of force mains; the selection was based on a judgment of each force main's potential to experience high surge pressures. The criteria used were the length and profile of the force main. Normally, long mains and profiles characterized by large elevation differences have greater surge potential as a result of abrupt changes in flow velocity.

The surge analysis was performed using the computer program LIQT version 6 developed by the University of Michigan. The program requires as input the estimated pressure wave speed in the force main, which is a function of the force main material; the force main's friction loss coefficient (C-value when using the Hazen-Williams equation); head-discharge data of pumps at the lift station, as well as the pumps' torque data; profile and size of the force main; and the flow rate through the force main. With these perimeters, simulation of power failure, which normally induces the most severe surge pressures, was simulated.

The likelihood of failure due to surge is based on the ratio of maximum surge pressure to rated pressure. These values were then normalized on a scale from one to five to determine a surge pressure rating.

7.1.2.2.6 Special Considerations

The model for this risk analysis was developed so as to be easily manipulated if other criteria are to be added. If there are any special considerations or other known variables that should be included in the analysis, the flexibility of this model provides the ability to add them quickly and easily.

7.1.3 FORCE MAIN RATINGS

As mentioned above, the average rating of consequence and likelihood factors were computed. The consequence and likelihood ratings were multiplied to determine the risk for each force main, relative to the other force mains, and the force main risk ranks were sorted into a priority list. Table 20 shows the results of this analysis. A color-coded rank was given to each force main based on Figure 58.

The tables used to determine likelihood and consequence ratings are available in Appendix I.

Table 20 - Force Main Risk Ranking

Force Main	Likelihood	Consequence	Risk	Material	Age	Length	Notes
West Avenue	3.00	3.12	9.35	CI	52.2	3301	1500' to be replaced.
Greenmeadow 2 (ends 594' from Greenmeadow 1)	2.22	3.50	7.78	DI	40.7	594	
Pebble Valley	1.78	4.20	7.49	DI	42.2	4154	
Heyer Dr 2 (ends 1822' from Heyer Dr 1)	1.95	3.70	7.22	DI	42.2	1822	
Burr Oak Boulevard 1 (ends 2004' from PS)	2.13	3.16	6.73	DI	39.9	2004	
General Electric	2.48	2.53	6.28	DI	26.4	5034	To be replaced in 2010.
Greenmeadow 1 (ends 924' from PS)	1.79	3.50	6.27	PVC	11.2	924	
Greenmeadow 3 (ends 1945' from Greenmeadow 2)	1.23	4.40	5.42	DI	26.6	1945	
Coneview	1.50	3.55	5.33	DI	33.2	2563	
Heyer Dr 1 (ends 835' from PS)	1.49	3.55	5.28	DI	16.2	834	
Greenmeadow 4 (ends 2327' from Greenmeadow 3)	1.23	4.20	5.17	DI	24.4	2327	
Ruben Drive 1 (ends 1524' from PS)	1.63	3.11	5.07	DI	22.9	1524	
Burr Oak Boulevard 2 (ends 3538' from Burr Oak 1)	1.53	3.19	4.90	CI	42.6	3538	
Northview Road	1.71	2.77	4.73	CI	42.2	713	
Milky Way 3 (ends 124' from Milky Way 2)	1.54	3.06	4.72	CI/DI	36.6	124	
Wal-Mart	1.62	2.87	4.66	DI	20.2	1201	
Sunset Drive	1.74	2.67	4.65	CI	46.3	3831	
Badger Dr 1 (ends 1305' from PS)	1.89	2.31	4.36	DI	28.2	1305	
Milky Way 1 (ends 814' from PS)	1.41	3.06	4.31	PVC	19.8	814	
Milky Way 6 (ends 242' from Milky Way 5)	1.41	3.06	4.31	PVC	19.8	242	
Milky Way 4 (ends 41' from Milky Way 3)	1.38	3.06	4.21	CI/DI	25.2	41	
Greenmeadow 5 (ends 3940' from Greenmeadow 4)	1.09	3.83	4.17	DI	15.7	3940	
Milky Way 2 (ends 31' from Milky Way 1)	1.33	3.06	4.06	CI/DI	21.8	31	
Milky Way 5 (ends 25' from Milky Way 4)	1.33	3.06	4.06	CI/DI	21.8	25	
Fox Point	1.14	3.55	4.06	PVC	24.2	8160	
Ruben Drive 3 (ends 3850' from Ruben Drive 2)	1.23	3.24	3.98	DI	26.5	3850	
MacArthur Road	1.30	2.99	3.87	DI	21.4	2279	
Ruben Drive 2 (ends 1137' from Ruben Drive 1)	1.22	3.11	3.78	DI	25.7	1137	
Springbrook	1.26	2.94	3.70	DI	17.2	4056	
Corporate Drive 2 (ends 1323' from Corporate Dr 1)	1.38	2.47	3.40	PVC	13.2	1323	
Corporate Drive 1 (ends 3937' from PS)	1.81	1.78	3.21	PVC	9.2	3937	
Summit Avenue	1.10	2.71	2.99	DI	13.2	2324	
Hollidale	1.19	2.50	2.98	CI	28.2	68	
Woodfield	1.09	2.49	2.72	DI	23.7	701	
Corporate Drive 3 (ends 411' from Corporate Dr 2)	1.17	2.29	2.66	PVC	9.2	411	
Wesley Drive	1.05	2.47	2.59	PVC	10.2	1682	
Dana (River Hills)	1.02	2.48	2.53	PVC	9.2	1546	
Aviation Drive	1.08	2.25	2.42	PVC	12.16	4980	
West Bluemound	1.15	2.03	2.32	PVC	11.2	4732	
Badger Dr 2 (ends 3385' from Badger Dr. 1)	0.98	2.31	2.27	HDPE	1.7	3385	
Heritage Hills (Madison Street)	0.76	2.92	2.23	PVC	6.2	1816	
Tallgrass	0.97	2.07	2.00	PVC	12.8	1335	
Silvernail	0.89	2.18	1.94	PVC	9.2	3054	
Fox Lake Village	0.75	2.48	1.85	HDPE	4.2	3960	
Deer Path	0.74	2.47	1.81	PVC	9.2	1093	
Bluemound	1.18	1.48	1.74	DI	30.2	516	
River Place	0.58	3.01	1.74	PVC	17.2	405	
Rivers Crossing 1 (ends 1217' from PS)	0.53	2.52	1.33	PVC	11.2	1217	
Rivers Crossing 2 (ends 2649' from River Crossing 1)	0.40	2.52	1.00	PVC	2.3	2649	
Fiddlers Creek	0.40	2.45	0.99	PVC	9.2	1025	
Golf Road	0.38	2.46	0.93	PVC	27.2	1474	
Pearl Street 1 (ends 788' from PS)	0.35	2.31	0.81	PVC	2.2	788	
Pearl Street 2 (ends 648' from Pearl Street 1)	0.33	2.31	0.77	PVC	1.2	648	
Deer Trails	0.03	2.45	0.07	PVC	3.2	800	

Force mains being considered for elimination   Force mains schedule for elmination

7.1.4 ANALYSIS

The force mains made of ferrous material rise to the top of the list. This happens as a result of factoring corrosion into the likelihood of failure. Since plastics were assigned a corrosion rating of zero, they rank lower than the ferrous force mains.

Also, while the relative position is important when prioritizing force mains, the actual number, "risk rank" should not be interpreted as the *probability* of failure. However, when considering force mains in the yellow region for inspection, one may want to consider the relative rating of consequence and likelihood. If, for instance, the likelihood of failure is very high, but the consequence is low (given it a lower overall rating), one may need to weigh the importance of each of these factors. If failure in and of itself needs to be avoided at all costs, regardless of the consequence, one may want to consider those force mains with high likelihood ratings for further inspection.

In Table 20, the force mains highlighted in gray are already being considered for elimination, and those highlighted in light orange are scheduled for elimination. In addition, the City has completed the following projects:

- Pearl Street – Replaced with PVC force main.
- Badger drive – Replaced 3500' of force main with HDPE.
- Grey Terrace – This pump station was eliminated.
- Ruben Drive – One of two discharge force mains have been eliminated.
- Greenmeadow – Replaced 924' of force main with PVC.

There is a large gap between the highest rated force main (West Ave) and the second highest rated (a portion of Greenmeadow that starts approximately 924 feet from the pump station and ends approximately 1518 feet from the pump station [Greenmeadow 2]). West Ave received the highest likelihood of failure rating. With six documented leaks on a relatively short length of pipe, West Ave received the highest possible rating for "leaks per unit length". It is also one of the oldest pipes in the entire system, giving it a high "remaining life" rating. In addition, the West Ave force main lies in highly corrosive soils; 59% of the soil types in the area rated "high" for corrosivity, giving it one of the highest "embedment soil" ratings of all the force mains. The "operating pressure" rating did not particularly affect the overall rating and this force main was not selected as being susceptible to surge; however, the combined rating from "leaks per unit length", "remaining life", and "embedment soil" give this force main a very high likelihood of failure rating. This section of force main should be the first considered for physical inspection.

Again, it is important to note that this evaluation is primarily qualitative and based on readily-available data rather than field testing. A high ranking does not guarantee that an individual force main is in poor condition or that failure is imminent. The ranking is simply a tool for prioritizing the force mains that most warrant a physical inspection, from which actual remaining life might be inferred. The scoring and ranking is an inexpensive and reproducible method of prioritizing force mains for the far more expensive task of physical inspection and testing.

7.1.5 NEXT STEPS

Donohue recommends the City develop a force main testing and inspection program that utilizes the most appropriate and cost-effective technologies to use on the force mains at greatest risk of failure. The City will use this information in the preparation of a Capital Improvement Program, which will include force main replacement and/or rehabilitation.

7.2 CMOM PROGRAM PLANNING

A Capacity, Management, Operations, and Maintenance (CMOM) program is a documented set of best management practices intended to enable a collection system utility to operate in an efficient, reliable manner. Under Phase I of this project, Donohue has completed a preliminary gap analysis to identify potential areas for improvement in the City's operations and maintenance procedures. This analysis is largely in response to a letter dated October 14, 2008 to the City from the United States Environmental Protection Agency (EPA), which recommended that the City undertake a CMOM program. The letter included a Sanitary Sewer System Inspection that EPA conducted on May 13 and August 26, 2008.

EPA's document, *Guide for Evaluating Capacity, Management, Operation, and Maintenance (CMOM) Programs at Sanitary Sewer Collection Systems*, was used to complete the analysis. This guide is meant to be used by sewer system owners, inspectors/reviewers for the EPA, and consultants. It provides the framework for a CMOM program and indicates program achievements that the EPA looks for when completing sanitary sewer system reviews.

As its name implies, a significant portion of a CMOM program involves evaluating and maintaining system capacity. Figure 61 illustrates the process by which this project intends to evaluate system capacity. The SSES and Alternative Analysis tasks are works in progress to be substantially completed under Phase II.

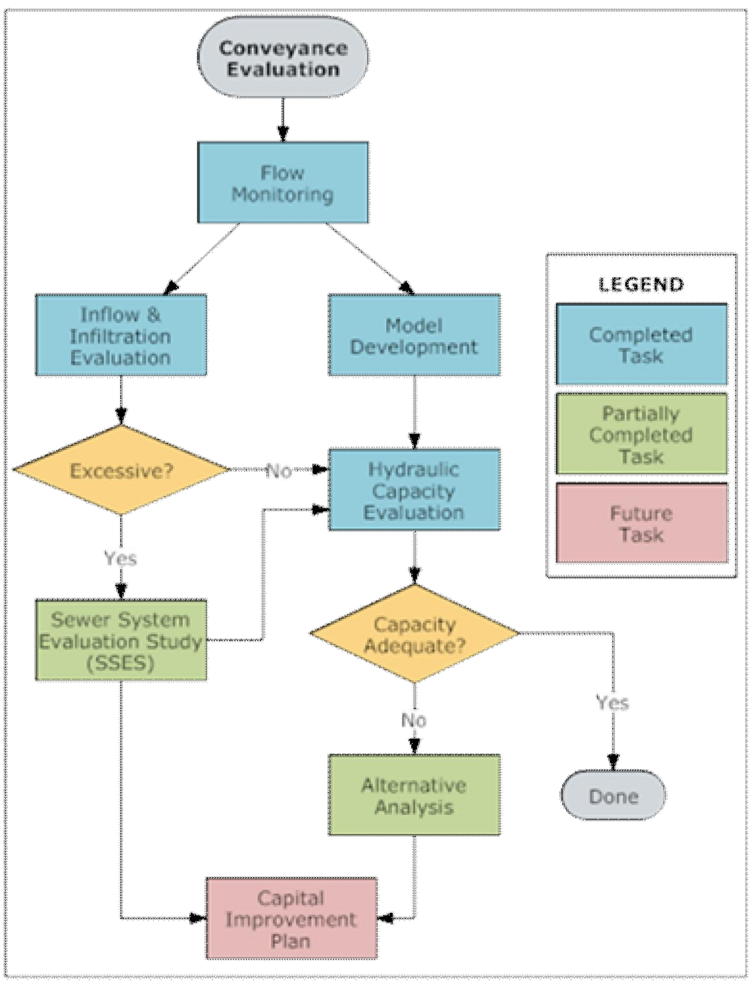


Figure 61 – Conveyance Evaluation Flow Chart

The City is already taking several proactive steps that satisfy elements of CMOM, including initiating the Sanitary Sewer System Master Planning project. In addition, the City is implementing a sewer televising program that will televise 10-15% of the sewer system per year. At this rate, the entire 250 miles of sewer will be televised every 7 to 10 years.

7.2.1 MANAGEMENT

7.2.1.1 Organization

The City's Public Works Department organization structure is as shown in Figure 62. The organization of the Public Works Department is well-delineated and conforms to CMOM recommendations. The engineering department is currently down 2 engineers and 1 tech position; however, those positions will likely remain vacant for 2010.

7.2.1.2 Training

The City currently has formal, documented safety programs covering several areas, including, lockout/tag-out, powered lift truck, fall protection, hazard communications, contractor safety, personal protection, emergency action plans, hearing conservation, etc. All programs are documented by the Wastewater Department. There is also annual field training for confined space entry, and classroom training that occurs approximately once every two years. The City is currently modifying the program to be consistent with OSHA standards.

Training for SSO's and emergency procedures currently exist, though informally. The Public Works Department ensures that employees understand what to do in the event of a power failure, for instance, but the procedures are currently undocumented. Documenting the current procedures will be a part of the CMOM program.

7.2.1.3 Communication

The Public Works Department consists of Engineering, Wastewater, and Streets Divisions. The City currently holds weekly meetings for the Wastewater Division, which is in charge of the pump stations. Streets is responsible for sewer cleaning and manhole maintenance, while Engineering is responsible for sewer televising, design, underground piping, major lift station upgrades / replacements. The structure is such that the separate Divisions can communicate effectively within their division as well as with one another.

The City also employs several methods of communicating with users and the general public. The website is updated frequently with upcoming projects. The website is also the primary means for the public to communicate with the Public Works Department. Emails received through the website go to the Director of Public Works and are routed to the appropriate division to respond. For specific projects, notification is sent directly to the affected residents via mailers or bill inserts sent with water utility bills. The local cable station, Channel 24, is also used for announcements, and the reverse 911 system can be used for emergencies.

7.2.1.4 Information Management

The City is currently in the process of evaluating several asset management software packages to integrate and modernize record keeping and centralize the information collected by different divisions. The Water Department uses the Azteca platform, and Public Works is currently tracking pump station repairs using MP2 software. The current practice for documenting sewer repairs is to mark up the as-built drawings. Any complaints received are also documented in a spreadsheet.

Selecting and implementing an asset management software package will be a critical step in the implementation of the City's CMOM program.

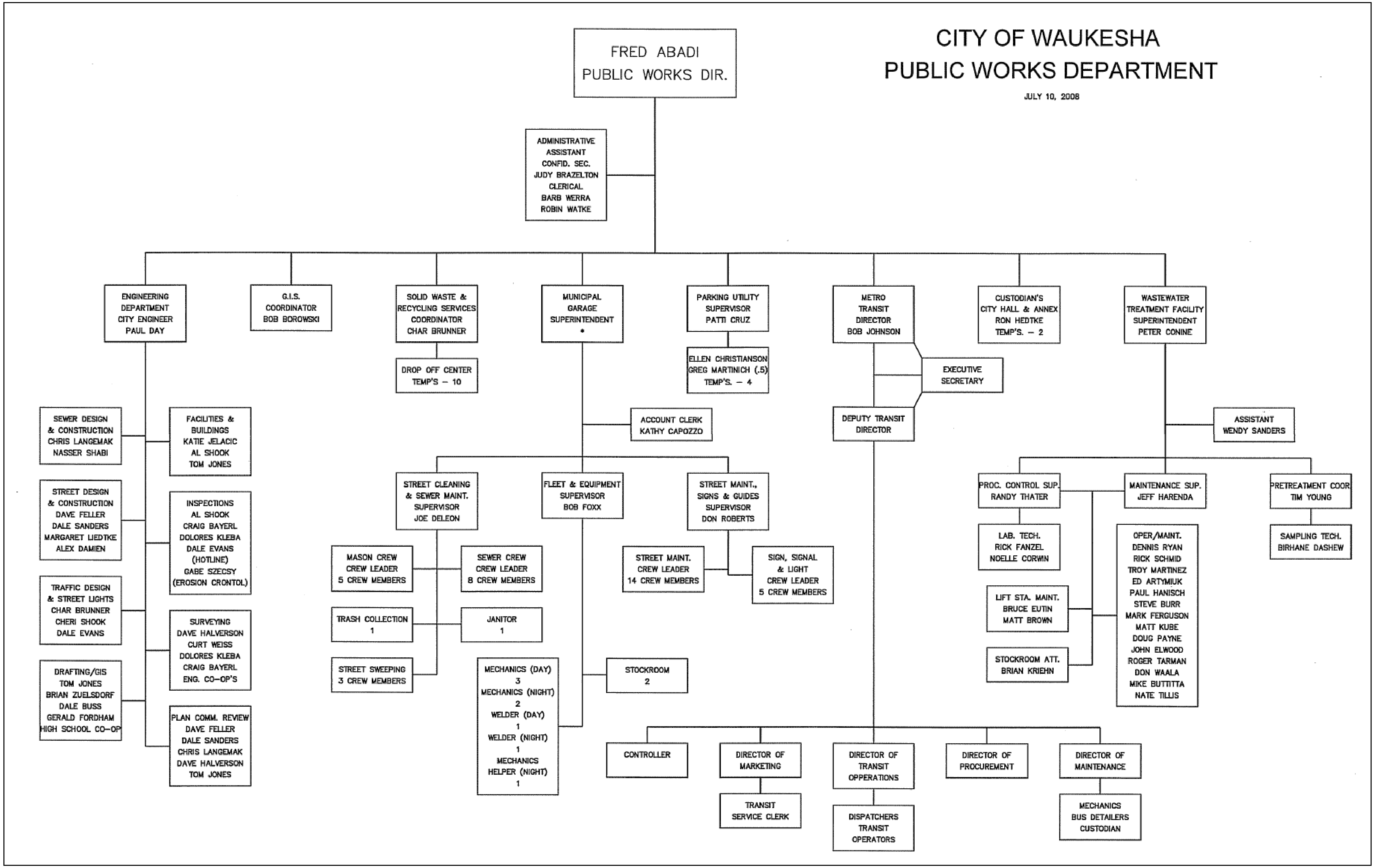


Figure 62 – City of Waukesha Department of Public Works Organization

7.2.1.5 Legal Authority

The City owns and operates its own collection system; there are no satellite communities from which it receives flow. Chapter 29 of the City of Waukesha Municipal Code addresses sewer use; it "sets forth uniform requirements for discharging wastes into the system, and enables the Control Authority to protect public health in conformity with all applicable local, state, and federal laws relating thereto." The code has explicit objectives; namely "to regulate the construction and use of all sewers and drains connected to the system, to set limitations or restrictions on materials or characteristics of waste or wastewaters discharged to the system, to prevent the introduction of pollutants into the wastewater treatment facilities, and to recover from users… the share of the wastewater treatment facility's costs, and to provide funds for the operation and maintenance, debt service, replacement and improvements of the wastewater treatment facilities." The ordinance contains standards for inspection, pretreatment requirements, and building/sewer permit issues; it also prohibits all materials listed in the EPA guidance manual (fire and explosion hazards, fats, oils, grease, etc). The City has the authority, via this ordinance, to enact enforcement and/or impose penalties on users who violate the ordinance.

7.2.2 OPERATION

7.2.2.1 Monitoring

The wastewater department has a pre-treatment program for 36 industrial users; most of these are monitored and some are zero-discharge users. Those that are monitored are sampled twice per year by the City and self-monitor bi-annually. Samples are analyzed to ensure they comply with local and federal limits. There is also a program to monitor plant influent and effluent quarterly and semi-annually. Sampling records should include date, time, and location of sampling and sample parameters.

7.2.2.2 H_2S Monitoring

There are a few private pump stations that have odor problems, and there has been a failure of a concrete gravity main into which several pumping stations discharged. This was likely the result of scour and/or H_2S corrosion. The televising program is underway, the City is prioritizing the concrete gravity mains fed by pumping stations to try and detect H_2S defects prior to a failure.

The City may want to consider identifying other areas prone to H_2S buildup, such as

- Sewers with low velocity conditions and/or long detention times
- Sewers subject to solids deposition
- Turbulent areas, such as drop manholes or force main discharge points

I&I reduction could negatively impact the risk of corrosion by reducing sewer cleansing that accompanies wet weather flows. CMOM notes that "a system in which infiltration and inflow (I&I) has successfully been reduced may actually face an increased risk of corrosion. The [City]/reviewer should pay particular attention to the hydrogen sulfide monitoring program in these systems." A desktop sewer velocity evaluation could identify those sewers at risk of low velocities, sediment deposition, and corrosion.

7.2.2.3 Emergency Response

With the exception of a few grinder stations, each pumping station has a SCADA alarm system; City personnel are on call for any emergencies. The City also owns and maintains four portable generators and two permanent generators (at Greenmeadow and Ruben Drive Pumping Stations). A permanent generator is being added to Summit. Permanent generators will also be installed at new and/or critical stations.

As part of this project, Donohue completed a desktop risk assessment of the City's force mains, several of which have experienced chronic leaks and/or failures. This risk assessment considered several factors in terms of the likelihood and consequence of a failure and ranked each force main for inspection accordingly. Phase II will include physical condition assessments of the force mains and establish a rehabilitation and maintenance schedule.

Force main corrosion has been a significant problem, though the City has been replacing ferrous pipes (ductile iron and cast iron) with plastic ones, typically PVC. Approximately 44% of the force mains in the City's collection system are now plastic.

The City has scheduled two force mains for elimination and is considering eliminating several more. The Grey Terrace pump station and one of the Ruben Drive force mains have already been eliminated.

The City coordinates with the WWTP department and Streets Department to respond to force main failures. The WWTP department is responsible for the pump stations and ensuring that flow is no longer routed to the effected force main. The municipal garage is capable of limited repairs; anything outside the scope of their services is contracted out.

7.2.2.4 Modeling

The EPA considers sanitary sewer modeling an important aspect of the CMOM program to determine capacity requirements, model before and after scenarios (with respect to rehabilitation, for instance), and predict future flows in portions of the collection system. As part of this project, the City has undertaken extensive modeling of their collection system. Donohue developed and calibrated a model using MikeUrban/MOUSE 2009. The model is currently being used to analyze the feasibility of eliminating up to eleven pump stations and evaluating system capacity. In Phase II, the model will be used to approximate future flows to determine improvements needed to provide reliable wastewater collection and treatment for current and future customers.

7.2.2.5 Mapping

Current and accurate maps of the collection system are another important aspect of CMOM that can assist with asset management. The City currently maintains a complete and accurate GIS database that includes the following components (as listed in EPA's CMOM guidance manual):

- Main, trunk, and interceptor sewers
- Building/house laterals
- Manholes
- Cleanouts
- Force mains
- Pump stations
- service area boundaries,
- Roads, water bodies, etc

The City is also diligent about updating the map whenever new construction takes place or if any system components change. The map is currently being used to assist maintenance crews. The data contained in the GIS database was also used for the Force Main Risk Assessment.

The map is a useful tool for tracking system maintenance and repairs; the City is currently evaluating how to integrate the GIS with asset management software so as to fully take advantage of Computerized Maintenance Management System (CMMS) technology to manage a more efficient and reliable operation.

7.2.2.6 New Construction

For new construction the City provides conveyance inspection during installation. Their design standards reference the Standard Specifications for Sewer and Water Construction in Wisconsin, 6th Edition. To confirm available capacity, the City looks at the entire service area and accounts for future development when reviewing construction plans.

7.2.2.7 Pump Stations

The City owns and maintains 45 pumping stations in their collection system. Pump stations are inspected weekly and a sign-off sheet is kept at each station for the inspector to log their visit. There is also an annual maintenance schedule; inspectors fill out forms for each pump station for this maintenance and make notes in plant operator logs. The City has two full time personnel dedicated to pump station operation and maintenance.

A SCADA system is used to monitor pump stations, though it does not provide remote operation. Approximately 50% of the pumping stations have O&M manuals stored at the Wastewater Treatment Plant.

7.2.3 MAINTENANCE

7.2.3.1 Planned & Unplanned Maintenance

This element of the CMOM program that will require the most effort for the City to meet program guidelines. CMOM describes maintenance using four categories: predictive, preventive, corrective, and emergency. Predictive and preventive maintenance are considered planned maintenance, while corrective and emergency maintenance are unplanned. The goal of the CMOM is to move to a largely planned maintenance program and minimize unplanned maintenance.

Predictive maintenance refers to monitoring equipment for warning signs of failure, such as excessive vibration, dirty oil, leaks, etc. Preventive maintenance refers to routine maintenance for system components, such as lubricating parts and following manufacturer recommendations. Preventive maintenance should be done on a regular basis and follow a specified frequency schedule.

Corrective and emergency maintenance occurs in response to failure; when this happens, resources are diverted from planned maintenance. Responding to system failures is often costly and may have environmental and health/safety consequences. While unplanned maintenance cannot be avoided altogether, improved predictive maintenance should minimize these types of repairs and serve to better preserve the collection system and improve budget forecasting.

The key component of maintenance, like most of CMOM, is documentation. CMOM recommends categorizing maintenance so as to track utilization of City resources. Maintaining set schedules and keeping information in a CMMS will better ensure that maintenance is conducted efficiently and effectively.

Recording and tracking maintenance will be greatly improved once the asset management software is fully implemented and integrated with the City's extensive GIS database. Utilization of these two software platforms in concert will be instrumental for efficient collection system maintenance and management.

7.2.3.2 Sewer Cleaning

The entire 250-mile sewer system is typically cleaned annually by four crews, each of which is assigned to a specific quadrant of the City. The cleaning crews also identify areas with grease buildup or root intrusion and document these areas on inspection reports. Cleaning consists of hydraulically flushing the sewers; no chemical cleaning is done. Some areas that have chronic root or grease problems are on 30- or 60-day maintenance/cleaning schedules.

The cleaning crews keep records of which sewers have been cleaned. Sewer cleaning records and inspection reports will be reviewed in Phase II to determine how sewer cleaning can be better integrated into the maintenance/inspection program.

7.2.3.3 Parts & Equipment Inventory

The City keeps spare pumps, check valves, level sensors, relays, starters, and other failure prone parts on hand. There is no written inventory; the current practice is to simply order a new part if one is used.

CMOM recommends keeping a written inventory of spare parts that includes the following information:

- Type, age, and description of the equipment,
- Manufacturer,
- Fuel type and other special requirements, and
- Operating costs and repair history.

7.2.4 SUMMARY

The City is currently utilizing the following elements of CMOM:

- Department Organization,
- Communication,
- Legal Authority,
- Modeling, and
- Mapping.

Documentation is really the backbone of the CMOM program. Some work remains to close the gap between the City's current operation and some elements of CMOM. In Phase II, Donohue will work with the City to establish programs and/or documentation to better implement the following CMOM components:

- Training programs, monitoring programs, emergency response procedures, safety procedures, sewer cleaning, spare parts, planned and unplanned maintenance, and pumping stations;
- Information management and integration with GIS; and
- SSES program.

7.2.5 NEXT STEPS & PHASE II

Phase II of the Sanitary Sewer Master Planning project will include a review of current documentation procedures and forms, address implementing the CMOM procedures discussed above, and include a more comprehensive SSES.

Phase II will also build upon the CMOM Program Planning documented here, and result in the creation of a CMOM Implementation Plan. After conducting further analyses of City operations, Donohue will identify specific areas for improvement in the areas of operation, maintenance, rehabilitation, documentation, etc. Improved data information management techniques including upgraded forms and paperwork and the proper implementation of CMMS will be specified, The CMOM Implementation Plan will contain a specific set of recommendations by which the City will have a program that is consistent with Federal guidelines.

CHAPTER VIII –MASTER PLANNING COST SUMMARY

The table below summarizes the preliminary costs developed thus far. These will be expanded and refined under Phase II of this project.

Description	Probable Cost
Pump Station Flood Protection	
Aviation Drive	$26,000
Coneview	$36,700
Pebble Valley	$55,600
Summit	$64,000
Sunset	$66,100
Sub-Total	$248,400
West-Side Bypass	$11,000,000
Southeast Bypass	$6,850,000
Fox Point Pump Station	T.B.D.
Total	**$18,098,400**

CHAPTER IX – LOOKING AHEAD – PHASE II

9.1 SEWER SYSTEM EVALUATION SURVEY (SSES)

9.1.1 PROGRAM DEVELOPMENT

A complete SSES program will be part of Phase II of this Sanitary Sewer Master Plan. Physical testing and inspection will identify and document problem areas and serve as a collection system baseline condition assessment. While the general elements of this program are described here, the preparation of a detailed program plan will be one of the first tasks to be completed under Phase II.

9.1.2 FLOW MONITORING & I&I QUANTIFICATION

During the spring and summer of 2009 portable flow meters were installed at several locations to quantify infiltration and inflow and to calibrate the model. Donohue recommends Phase II flow monitoring at the three locations in the Heyer Dr area of the City, which experiences excessive I&I. While the City does not maintain any permanent flow meters, the pumping stations' SCADA system can be used to continue to monitor flow.

9.1.3 BUILDING INSPECTIONS

While flow monitoring revealed the presence of significant inflow in the Pebble Valley service area, particularly the portion monitored by meter #18, smoke testing found few defects. Gravity foundation drains and/or sump pumps are the most likely source of I&I. Physical building inspections are the most reliable means of testing for the existence of these illicit connections, however these can be disruptive to the homeowner.

Under Phase II of this project, Donohue will work with City personnel to conduct building inspections in a pilot area TBD. Donohue will coordinate with the City to develop an inspection protocol that minimizes property owner inconvenience.

9.1.4 SMOKE TESTING

Due to its low cost and success in locating defects in the downtown area, Donohue recommends that Waukesha smoke test the remainder of this area in Summer 2010. The area to be tested is likely to be approximately 2,000 acres in size containing approximately 48 miles of sewer with 75% of those over 50 years old. The cost to test this area is approximately $75,000, however the City need not test it all in one year.

9.1.5 MANHOLE INSPECTIONS

While only a very limited portion of the collection system is visible from the surface, manholes can be a barometer of overall sewer condition. During Phase II, manhole inspections are likely to be conducted as a preliminary assessment of sewer condition. A standardized inspection procedure and data collection form will be developed such that the results can be incorporated into the City's asset management program.

9.1.6 SEWER TELEVISING / DYED-WATER FLOODING

The City plans to implement a sewer televising program that will televise 10-15% of the sewer system every year. At this rate, the entire 250 miles of sewer will be televised every 7 to 10 years. While additional flow monitoring in Spring 2010 will better isolate specific areas in the Heyer Drive service area contributing excessive I&I, Donohue recommends that Waukesha conduct spring sewer televising and perhaps dyed-water-flooding of those sewers where excessive infiltration is most likely originating. This would include older sewers, sewers more likely submerged by groundwater, and/or sewers crossing or adjacent to surface waters (creeks, ditches, etc.).

Smoke testing located areas of questionable sewer structural integrity in the Heyer Drive and downtown areas (Section 5.1.2). Donohue recommends that CCTV and dyed-water flooding be employed in these areas to locate specific defects.

9.2 FORCE MAIN CONDITION ASSESSMENT

Section 7.1 ranks each City force main according to its risk of failure. However this ranking is not a measure of actual force main condition. Donohue recommends that the City implement a force main physical inspection program to assess the true condition of those force mains at the greatest risk of failure.

There are a variety of technologies that could be brought to bear for this testing. While the application of several of these technologies to other industries is well established, most are in their infancy with regard to sanitary sewer force main testing. Perhaps the most proven technology is External Corrosion Direct Assessment (ECDA), however this can be rather expensive. Some other technologies that should be considered include, but are not limited to: "C-factor" testing, acoustic leak detection, etc.

9.3 FUTURE EXPANSION OF COLLECTION SYSTEM

The CS hydraulic model was developed with the intent of developing a 5-year Capital Improvement Plan (CIP). This plan will need to consider what improvements must be made to accommodate City growth, development, and re-development. Under Phase II of this project, Donohue will engage the City's Planning Department to gain a better understanding of land use, zoning, and potential for growth. Future flows will be estimated, and the skeletal sewer system expanded to serve areas of growth. The impact of this expansion and additional flow will be considered during the preparation of the CIP.

9.4 CAPITAL IMPROVEMENT PROGRAM

The principal deliverable of Phase II of this project will be a 5-year CIP. This plan will specify those capital improvement projects that the City will need to implement to continue to provide reliable service to current and future customers. Cost estimates of all recommended improvements will be prepared. Dates by when the recommended improvements should be implemented will be prepared so that the City can plan its capital budget accordingly.

9.5 CMOM IMPLEMENTATION PLANNING

A Capacity, Management, Operations, and Maintenance (CMOM) program is a documented set of best management practices intended to enable a collection system utility to operate in an efficient, reliable manner. Under Phase I of this project, Donohue has completed a preliminary gap analysis to identify potential areas for improvement in the City's operations and maintenance procedures. This analysis is largely in response to a letter dated October 14, 2008 to the City from the United States Environmental Protection Agency (EPA), which recommended that the City undertake a CMOM program. The letter included a Sanitary Sewer System Inspection that EPA conducted on May 13 and August 26, 2008.

EPA's document, *Guide for Evaluating Capacity, Management, Operation, and Maintenance (CMOM) Programs at Sanitary Sewer Collection Systems*, was used to complete the analysis. This guide is meant to be used by sewer system owners, inspectors/reviewers for the EPA, and consultants. It provides the framework for a CMOM program and indicates program achievements that the EPA looks for when completing sanitary sewer system reviews.

As its name implies, a significant portion of a CMOM program involves evaluating and maintaining system capacity. Figure 61 illustrates the process by which this project intends to evaluate system capacity. The SSES and Alternative Analysis tasks are works in progress to be substantially completed under Phase II.

CHAPTER X –BIBLIOGRAPHY

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APPENDICES

APPENDIX A – PUMP STATION FLOOD PROTECTION

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